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COSTING OF THE REVISED CANADIAN MAGLEV GUIDEWAY DESIGN

prepared for
Transportation Development Centre

July 1986

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17. Abstract

The dominance of guideway-related capital cost elements in the initial cost for the Canadian Maglev System (CMLS) makes this area of prime importance for cost reduction through design refinement and optimization. To test the possible effects of such design changes, and to obtain a higher degree of confidence in the associated cost estimates, CIGGT prepared two beam designs based on the revised system concept proposed by Hayes and Tucker, then obtained "commercial" cost estimates for large-scale production of each new design from three major precast concrete manufacturers. To provide a datum for comparison, the manufacturers also prepared cost estimates for the original CMLS beam design. The cost estimates and an analysis of the inputs of the design alternatives on system first cost are reported.

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17. Résumé

Comme la construction de la voie de sustentation accaparerait la plus grande part des coûts de premier établissement du système canadien de transport à sustentation magnétique, c'est le domaine où l'on pourrait réaliser les économies les plus intéressantes grâce à l'amélioration et l'optimisation de la conception. Dans le but de vérifier les effets possibles de tels changements de conception et d'avoir une idée plus précise des coûts connexes, le CIGGT a préparé les plans de deux modèles de poutres d'après le système proposé par Hayes et Tucker. Il a ensuite obtenu une estimation "commerciale" pour la production à grande échelle de chaque modèle par les trois principaux fabricants d'éléments en béton préfabriqués. À des fins de comparaison, les entreprises ont également fait des estimations pour le projet original. Le rapport présente ces estimations de coûts et analyse l'incidence du choix de l'un ou l'autre des modèles proposés sur le coût et premier établissement du système.

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Système canadien de transport à sustentation magnétique, voie, rail, coût	Disponible sur demande auprès du CDT ou de CIGGT

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SUMMARY

Since 1980, the original Canadian Maglev Group conceptual design described in "Alternatives to Air: A Feasible Concept for the Toronto-Ottawa-Montréal Corridor" (CIGGT Report No. 80-4, TP2667E, July 1980) has been extensively redesigned by W.R. Hayes and H.G. Tucker of the Division of Mechanical Engineering, National Research Council of Canada. These researchers postulated that design changes to the guideway and guideway-mounted levitation, propulsion and control components would offer the prospect of substantial reductions in the capital costs of a Maglev systems, as well as improvements in the system's operational reliability, maintainability, and safety, and a reduction in the specific energy consumption.

The objective of this project was to develop construction specifications and commercial cost quotations for the manufacture of the revised guideway design, and, for reference, equivalent cost estimates for the original design.

Design drawings and specifications were prepared for the original design and for each of the revised design options, and submitted to three qualified large-scale manufacturers of precast concrete -- Schokbeton Québec Inc. of St. Eustache, Québec, Pre-Con Limited of Brampton, Ontario, and Supercrete Ltd of Richmond, B.C. -- for commercial "factory gate" price estimates.

These commercial quotes were then analyzed to identify sources of variance among quotes and assess the impact of the quoted prices on the capital costs of a Maglev system.

Based on the findings of this study, it was concluded that both the box-section and channel-section beam designs would meet all technical and regulatory requirements for a Maglev system. Conventional (magnetic) steel reinforcements can be used in construction of the guideway beams without causing adverse effects on vehicle operations or on guideway life expectancy. The channel section would cost between 10 and 25 per cent less to fabricate than the box section. The channel section would also offer significant cost savings over the original trapezoidal trough/top slab design. Compared to the corresponding revised costs for the original design, the channel section would permit savings in the range of 15 to 35 per cent. The Schokbeton price levels for the channel section correspond to a savings of about 42 per cent (including the savings from fewer support structures), as compared to the revised estimates of the original beam design. The use of epoxy-coated reinforcing bars would increase the cost of the guideway girders by 10 to 20 per cent.

SOMMAIRE

Depuis 1980, le concept original imaginé par le Groupe canadien d'études sur la sustentation magnétique et décrit dans "Alternatives to Air: A Feasible Concept for the Toronto-Ottawa-Montréal Corridor" (Rapport du CIGGT #80-4, TP2667E, juillet 1980), a été considérablement remanié par W.F. Hayes et H.G. Tucker de la Division du génie mécanique, Conseil national de recherches du Canada. Ces chercheurs ont considéré qu'en modifiant la voie et les éléments de sustentation, de propulsion et de commande installés dans la voie, on pourrait réduire substantiellement les coûts d'investissement d'un système de transport par sustentation magnétique, améliorer la fiabilité, la maintenabilité et la sécurité opérationnelle du système et aussi réduire la consommation d'énergie spécifique.

Ce project avait pour objet l'élaboration des plans et devis d'une voie de configuration nouvelle et l'obtention de propositions de prix commerciaux pour la fabrication de ses éléments et, à des fins de référence, l'estimation des coûts équivalents pour le projet original.

Des plans et devis ont été préparés pour le projet original et pour chacune des variantes proposées et soumis à trois entreprises reconnues de fabrication à grande échelle d'éléments en béton préfabriqués -- Schokbeton Québec Inc., St-Eustache, Québec; Pre-Con Limited, Brampton, Ontario; et Supercrete Ltd, Richmond, C.-B. -- en vue d'obtenir des estimations de prix commercial "à la sortie de l'usine".

Ces trois propositions ont été ensuite analysées en vue de cerner les écarts et d'évaluer l'incidence des prix proposés sur les coûts d'investissement d'un système de transport par sustentation magnétique.

Après étude, les spécialistes ont conclu que les variantes à poutres-caissons et à poutres U répondraient à toutes les exigences techniques et réglementaires d'un système à sustentation magnétique, et que des armatures d'acier classiques (magnétiques) pourraient être utilisées dans la construction des poutres sans nuire au fonctionnement du véhicule ni réduire la durée de vie prévue de la voie. La fabrication des poutres U coûterait entre 10 et 25 p. 100 moins cher que celle des caissons. La poutre U permettrait également de réaliser des économies substantielles par rapport au modèle original à auge trapézoidale/dalle supérieure. Par comparaison avec les coûts révisés du concept original, la poutre U permettrait de faire entre 15 et 35 p. 100 d'économies. Les prix établis par Shockbeton pour la poutre U correspondraient à des économies d'environ 42 p. 100 (incluant les économies réalisées par l'utilisation d'un nombre réduit de structures de support), par rapport aux estimations révisées du projet original à poutres. L'utilisation de barres de renfort enduites de résine époxy augmenterait de 10 à 20 p. 100 le coût des poutres constituant la voie.

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1. INTRODUCTION

1.1 Background

In 1980, a team from CIGGT completed, on behalf of the Transportation Development Centre, a comprehensive systems analysis[1] of the electrodynamic Maglev design developed by the Canadian Maglev Group, a 260 km/h high-speed rail system, a 200 km/h intermediate-speed rail system, and air as the existing high-speed mode, for service in the Toronto-Ottawa-Montréal portion of the Central Canada corridor. This study concluded that although Maglev could be economically attractive under some scenarios, a more conventional high-speed rail system would be superior under the marginal traffic conditions which prevail in this corridor.

This study also identified the guideway and guideway-mounted levitation, propulsion and control components as the dominant capital-cost elements, accounting for about 75 percent of the total Maglev system cost. In contrast, the cost of the Maglev vehicles, even incorporating the estimated development engineering and prototype construction and testing costs, and substantial contingency allowances, amounted to only 21 percent of the total capital cost.

Clearly, the guideway and guideway-mounted components offer the greatest opportunity for cost savings through design improvements. Such first-cost reductions would be particularly effective in improving the commerical viability of the system.

The guideway and (most of) the guideway-mounted components also represent the most "conventional" aspects of this advanced-technology system. Potential investors -- public or private -- consequently should be expected to treat the cost estimates associated with these elements as "hard" data in any risk analysis.

Since 1980, the original Canadian Maglev Group conceptual design has been extensively redesigned by W.F. Hayes and H.G. Tucker of the

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1. Lake, R.W., et al. "Alternatives to Air: A Feasible Concept for the Toronto-Ottawa-Montréal Corridor." CIGGT Report #80-4, July 1980.

Division of Mechanical Engineering, National Research Council of Canada.[2] These researchers postulated that the proposed design changes would offer the prospect of substantial reductions in system first-cost, as well as improvements in the system's operational reliability, maintainability, and safety, and a reduction in the specific energy consumption.

1.2 Objectives

This project has three overall objectives:

- to improve the cost data base for Maglev systems to enable a more effective assessment of commercial potential, and thus to provide guidance for establishment of research priorities for Maglev technology in Canada
- to maintain and enhance Canada's status as a leading practitioner in the area of comparative systems costing and analysis
- to improve Canadian knowledge of cost-effective precast concrete guideway design with a view to the potential value of this knowledge to highway, transit and grade-separation applications as well as to advanced systems

and one specific objective:

- development of construction specifications and commercial cost quotations for the manufacture of the revised guideway conceptual design, and, for reference, equivalent cost estimates for the original guideway design.

1.3 Approach

In addressing these objectives, the CIGGT team has focused on the most critical element identified in "Alternatives to Air" -- guideway

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2. Hayes, W.F., and H.G. Tucker. "Design Concept and Comparative Performance of an Electrodynamic Maglev Transportation System for the Toronto-Montréal Corridor of Canada." National Research Council of Canada. IMechE, C400/84, 1984, pp. 137-148.

capital cost -- while retaining the basic system defintion and configuration.

Appropriate design drawings and specifications were prepared for the "Alternatives to Air" design and for each of the revised design options, and submitted to three qualified large-scale manufacturers of precast concrete -- Schokbeton Québec Inc. of St. Eustache, Québec, Pre-Con Limited of Brampton, Ontario, and Supercrete Ltd of Richmond, B.C -- for preparation of commercial "factory gate" price estimates.

These "commercial quotes" were then analyzed to identify (where possible) sources of variance among "quotes," and to assess the impact of the "quoted" prices on system first cost.

1.4 Report Organization

Section 2 presents a brief summary of the baseline system definition from "Alternatives to Air." Section 3 describes the "Alternatives to Air" reference beam design and discusses the development of the "commercial quotes" for that design. Section 4 presents the revised beam designs and associated "commercial quotes." Section 5 contains the discussion of results, while Section 6 presents conclusions and recommendations. Technical details are presented in the accompanying appendices.

2. BASELINE SYSTEM DEFINITION

The baseline Maglev system analyzed in "Alternatives to Air" was configured to accommodate an initial (1995) traffic volume of about eight million Toronto-Montréal one-way trip equivalents, growing to almost 16 million by 2025. The system routing, illustrated in Figure 2.1, served Toronto, Kingston, Ottawa, Mirabel Airport and Montréal.

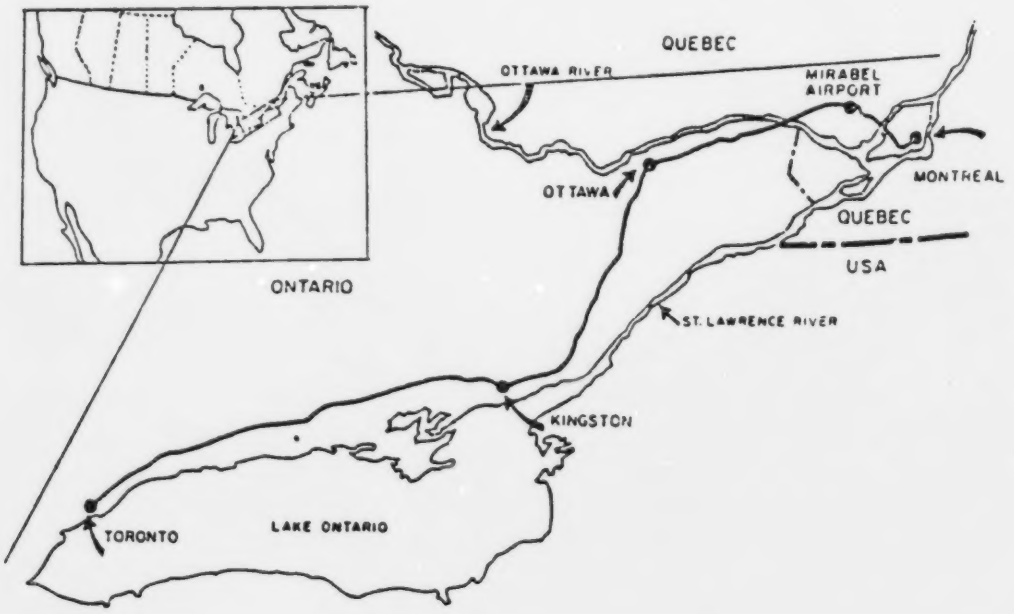

Figure 2.1 Toronto-Montréal corridor routing for the 450 km/h Maglev system

The route began at the proposed Toronto terminal on the site of the old CP station on Yonge Street, next to the Summerhill subway station. The selected routing then followed an existing CP right-of-way out of Toronto, passing south of the major marshalling yards. East of Brock Road, the alignment ran to the northeast, ascending the south face of the Oak Ridges moraine. The route then swung eastward, passing south of Rice Lake, then turned to the northeast to cross the Trent River near Frankford. From that point, the alignment ran east, passing just south of Varty and Loughborough Lakes. The Kingston station was located near Elginburg, on Frontenac County Road No. 9. From this point, the route skirted the south side of Collins Lake, crossed the Cataraqui River near Washburn and the Gananoque River

south of Marble Rock Dam, then turned to the north, skirting the southeast corner of the Charleston Lake region. This alignment exited the Frontenac Axis just south of Athens, then turned to the northeast south of Merrickville, crossing the Rideau River near Kemptville. After passing east of North Gower, the alignment recrossed the Rideau near Manotick, then proceeded east onto the southeast corner of the Ottawa International Airport property, where the Ottawa Station was to be located.

From Ottawa, the route ran just to the north of Carlsbad Springs, proceeding south and east across the Ottawa Valley clay plains. The Ottawa River was to be crossed at Hawkesbury, using a 1,100-metre span. The route then ran east-southeast, passing to the south of Saint-Phillipe-d'Argenteuil, crossing the Rivière du Nord, then proceeding to the northeast to the vicinity of Saint Canut. At this point, the route curved sharply to the south to pass in tunnel under Mirabel Airport, then resumed its eastward alignment as far as Saint-Augustin. The route then turned to the southeast, along the east side of Highway 13, and followed that alignment across both Rivière des Milles Iles and Rivière des Prairies. The route skirted the northeast corner of Dorval Airport, crossed the southwest corner of the Cote Saint-Luc marshalling yards, then followed the north side of the existing CN Rail right-of-way into downtown Montréal. The proposed Montréal terminal was to be located on the site of the old Bonaventure goods station.

The system length was 592 kilometres.

As analyzed for "Alternatives to Air," the basic guideway structure for the Maglev system consisted of a series of 25 metre precast, post-tensioned, reinforced concrete box girders, supported by precast, post--tensioned, reinforced concrete pier supports. The design mean elevation above grade was about 7 metres.

To accommodate the short headways (5 minutes) required to handle forecast traffic levels during peak periods, separate eastbound and westbound guideways, separated by at least 3 metres to reduce the aerodynamic effects on passing vehicles, were costed. Most of the route was costed with two beams supported on a single cross-head and column. Where geometric constraints were not compatible with this

approach, as at intermediate stations or in some urban rights-of-way, a single beam/single column guideway was costed.

CIGGT noted at that time that "the structural system described herein only represents a likely competitive bid for a design and build contract for the guideway. Alternative structural concepts would presumably be proposed in accordance with the special expertise of the bidder. A firm or consortium with considerable experience in steel construction might therefore propose a composite steel and concrete structure, supported on steel rather than concrete piers. Naturally, any design would have to meet standards for electromagnetic interference and environmental disruption as well as the basic structural requirements." Nothing that has happened in the intervening years, including the investigations reported herein, has altered the validity of this statement.

3. DESCRIPTION AND COSTING OF THE "ALTERNATIVES TO AIR" REFERENCE DESIGN

The "Alternatives to Air" guideway structure was designed to emphasize field erection rather than construction, so as to take advantage of the economies of scale possible with the high-volume production of precast concrete components. This strategy would give the system developer additional flexibility in the scheduling of production and erection operations, since the finished components would be stockpiled for erection during the rather limited field season. The use of factory rather than field construction would also facilitate better quality control. The LSM windings, levitation strips, null-flux loops and positive guidance barriers would be installed as part of the factory assembly for this reason.

The original guideway design called for a series of 25 m long concrete beams supported at an elevation of approximately 7 m on reinforced concrete piers. For double-guideway construction, the beams could be supported either on a single pier with a cross-head, or on two individual piers.

Details of the beam reference design are shown in Figure 3.1. Each girder comprises a precast concrete trough beam section with a cast-in-place reinforced concrete top slab. Longitudinal prestressing is supplied by a combination of pre-tensioning and post-tensioning. The guideway was originally conceived as a series of five-span continuous girders, achieved by erecting five pretensioned trough sections as simply supported beams, pouring the in situ slab, and post-tensioning the five spans together. Subsequently, in order to reduce the size of expansion joints required in the guideway, it was decided to make the girders simply supported. This approach eliminates the need for in situ casting of the top slab and for multiple-span post-tensioning. The revised designs discussed in Section 4 also reflect this change.

Appendix A presents the original structural design analysis of the beam undertaken for "Alternatives to Air."

To develop costs for "Alternatives to Air," cost data for similar structures were obtained from several sources, including the Urban Transportation Development Corporation, a major Ontario-based precast

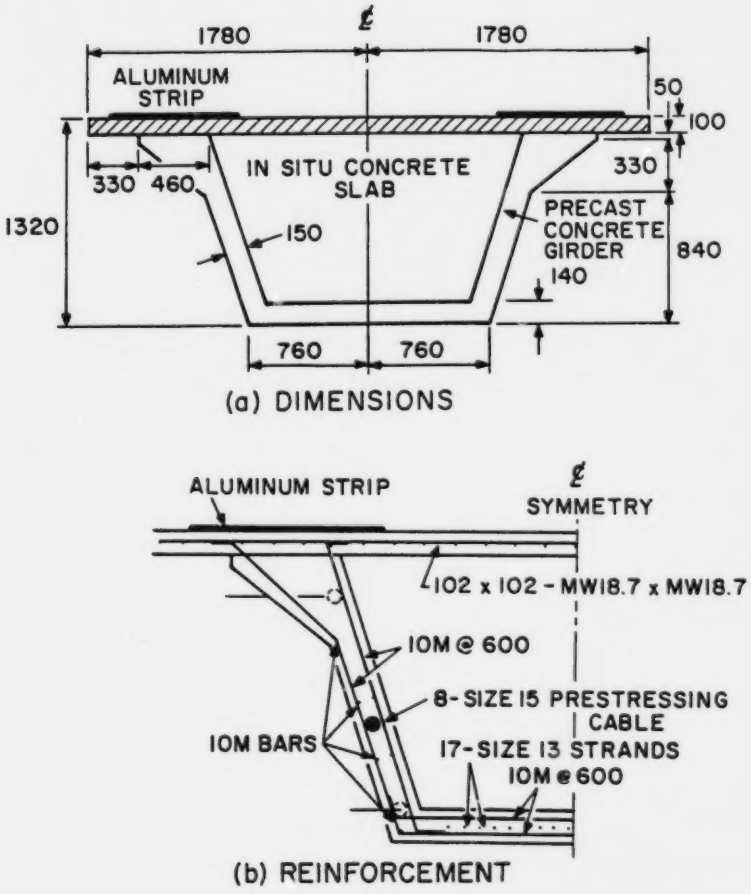

Figure 3.1 Details of original guideway beam design (dimensions in mm)

concrete manufacturer, then-current (1978) consultants' reports, and the Ontario Ministry of Transportation and Communications in Toronto. The OMTC provided considerable historical cost data drawn from construction contracts dealing with precast concrete beam bridges. The unit cost of $523/metre ($1978) developed for that study represented the authors' best estimates, based on a synthesis of these data. That represents a cost of about $873/linear metre in $1986, based on 67 percent increase in the relevant cost index.

In the current study, "commercial" cost estimates for the precast trapezoidal trough portion of the original beam were obtained from Schokbeton and Pre-Con. The cost of the in situ top slab and single-span post-tensioning was estimated by one of the authors (Dr. Campbell) from comparable industry data. These costs, which are

summarized in Table 3.1, are f.o.b. plant, and do not include federal or provincial sales taxes.

The very substantial variance in "quoted" unit costs for the original trough section is also apparent in the quoted estimates for the revised beam designs and is discussed further in Section 5.

TABLE 3.1

COMPARISON OF UNIT COST ESTIMATES
FOR GUIDEWAY BEAM REFERENCE DESIGN
($1986/linear metre)

			"Alternatives to Air"	
Component	$1978	$1986	Schokbeton	Pre-Con
Trough			$442	$1,436
Top slab/post-tensioning			340	340
Total	$523	$873	$782	$1,776

4. DESCRIPTION AND COSTING OF THE REVISED GUIDEWAY BEAM DESIGN

The revised beam design proposed by Hayes and Tucker is illustrated in Figure 4.1. This conceptual design served as a starting point for the further revisions presented in this section and its appendices.

The basic beam was redesigned as a pre-tensioned, precast, simply supported concrete girder capable of withstanding the loading conditions summarized in Table 4.1. The "worst case" loading would occur if a vehicle traversing a combined minimum radius lateral and vertical curve at cruise velocity were to drop onto its emergency skids in the presence of a design-limit cross-wind with maximum velocity directed radially outward relative to the curve. Service and strength load combinations used in the design are given in Tables 4.2 and 4.3, respectively. These combinations are an adaptation of those proposed

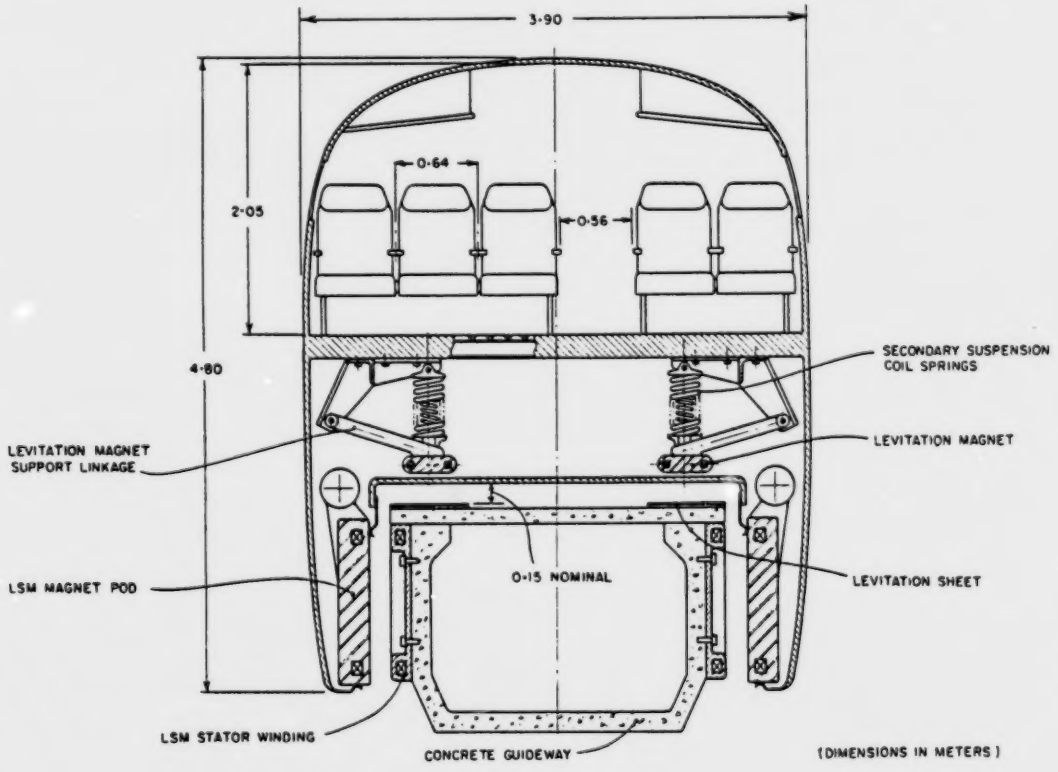

Figure 4.1 Proposed Maglev vehicle/guideway typical cross-section

TABLE 4.1

LOADINGS ON GUIDEWAY GIRDER

Guideway Loading Source	Static Loading*		Dynamic Loading	
	kN/m	kN/Span	kN/m	kN/Span
Vertical loading:				
. vehicle mass induced force	10.95	274.0	19.71	493.0
. vertical curvature induced centrifugal force	1.09	27.0	1.96	49.0
Total vertical loading	12.04	301.0	21.67	542.00
Lateral loading:				
. superelevated guideway centrifugal force	2.19	55.0	3.17	79.0
. cross-wind induced aerodynamic drag force	5.29	132.0	7.67	192.0
. levitation magnet destabilization force	1.81	45.0	2.62	66.0
Total lateral loading	9.29	232.0	13.46	337.0
Alternative longitudinal loading:				
. normal emergency braking force	3.28	82.0	3.28	82.0
. extreme emergency braking force	3.10	77.0	5.58	139.0
Overall resultant "worst case" maximum loading	15.56	389.0	26.12	653.0
Torque loading about longitudinal axis:			**	**
. superelevated guideway centrifugal force induced	3.94	98.5	-	-
. aerodynamic drag induced	8.46	211.5	-	-
. levitation magnet destabilization force induced	1.81	45.2	-	-
Total torque loading	14.21	355.2	-	-

* Excludes vehicle/guideway dynamic interaction effects.
** Analysis for torsional dynamic amplification factor not available (factor anticipated to be small due to the high torsional stiffness of the beam section).

(after Hayes and Tucker, 1984)

TABLE 4.2

SERVICE LOAD COMBINATIONS

$S1 = D + L + I + PS + LF_n + CF$

$S2 = S1 + 0.3\ WL$

$S3 = S2 + T + SH + CR$

$S4 = D + (WS\ OR\ EQ) + T + SH + CR$

D = dead load
L = live load
I = impact factor
PS = prestress
LF_n = normal longitudinal braking force
CF = centrifugal force
WL = wind load on live load
T = load due to temperature or thermal gradient
SH = forces due to shinkage of concrete
CR = forces due to creep of concrete
WS = wind load on structure
EQ = earthquake force

TABLE 4.3

STRENGTH LOAD COMBINATION

	Load Combination					
Load Component	U1	U2	U3	U4	U5	U6
D	1.30	1.30	1.30	1.30	1.30	1.30
-D	0.90	0.90	0.90	0.90	0.90	0.90
L, I and CF	1.40	1.40	1.40	1.40	1.40	
SH and CR	1.00	1.00	1.00	1.00	1.00	1.00
WL	1.50				1.50	
T or EQ		1.50				
LF_e			1.40			

LF_e = emergency longitudinal braking force

in a report by the American Concrete Institute Committee 358 -- Concrete Guideways.[3]

Both box-section and channel-section beam designs, shown in Figures 4.2 and 4.3, respectively, were considered. These sections were designed under the basic premise that the prestressing steel should be a minimum of 500 mm from the stator windings, and that other steel reinforcement should be kept to a minimum in these locations. Problems associated with magnetic reinforcement are discussed below. All girders were assumed to be straight since, for the minimum radius of horizontal curvature of 15.9 km, the maximum offset from a 25 m chord is less than 7 mm.

Design of the girders was carried out in accordance with CAN3-A23.3-MB4,[4] supplemented, where required, by requirements of the Ontario Highway Bridge Design Code.[5]

A concrete comprehensive strength of 40 MPa has been assumed together with a yield stress of 400 MPa for the non-prestressed reinforcement and an ultimate tensile strength of 1,860 MPa for the low-relaxation prestressed reinforcement. Debonding of some of the strands was utilized to control stress levels at the ends of the girders.

4.1 Design of the Box-Section Beam

Box-section girders have been used commonly in guideways to take advantage of their inherent torsional strength and stiffness. Figure 4.2(a) shows the basic dimensions of the box section. These dimensions are controlled largely by the configuration of the vehicle (as shown in Figure 4.1). The box section has an overall width of 2,600 mm, a wall thickness of 150 mm, and a depth of 1,350 mm. This is the minimum depth required to support the stator winding. Figure 4.2(b) shows the arrangement of the pre-tensioning strands, with the majority concentrated in the central zone of the bottom slab of the box. A

3. ACI Committee 358, "Analysis and Design of Reinforced Concrete Guideway Structures," Concrete International (in press).

4. CAN3-A23.3-M84, "Design of Concrete Structures for Buildings," Canadian Standards Association, Rexdale, Ontario, December 1984.

5. "Ontario Highway Bridge Design Code," Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

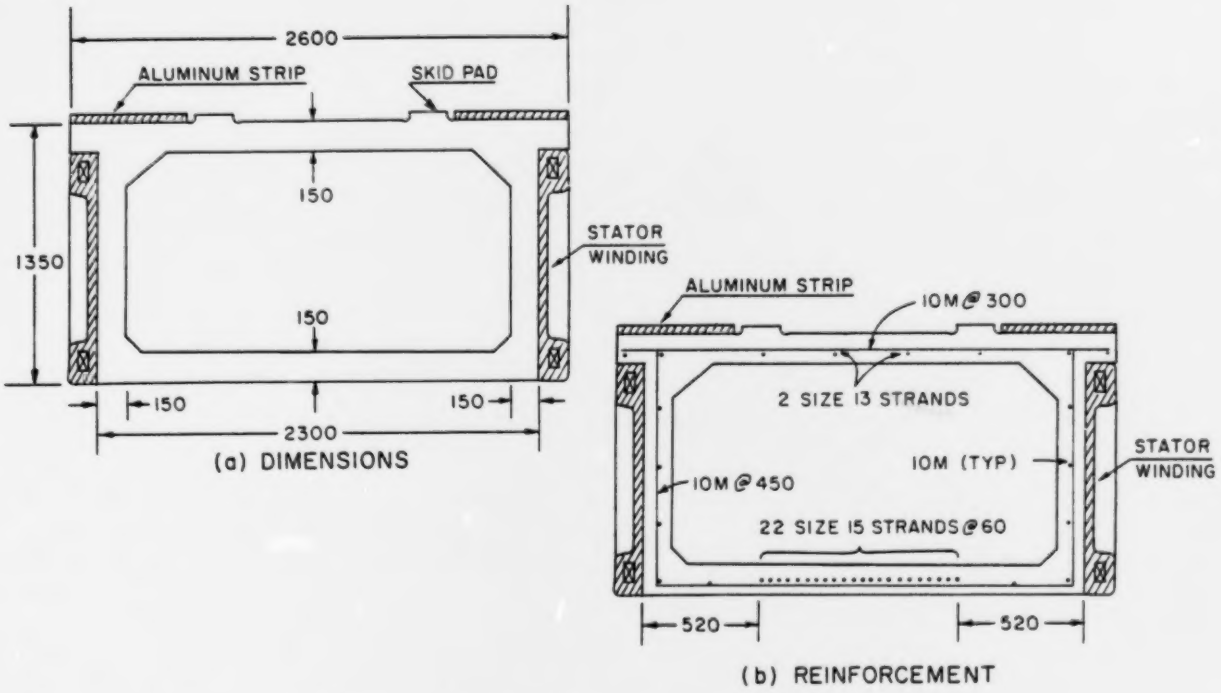

Figure 4.2 Details of box-section girder

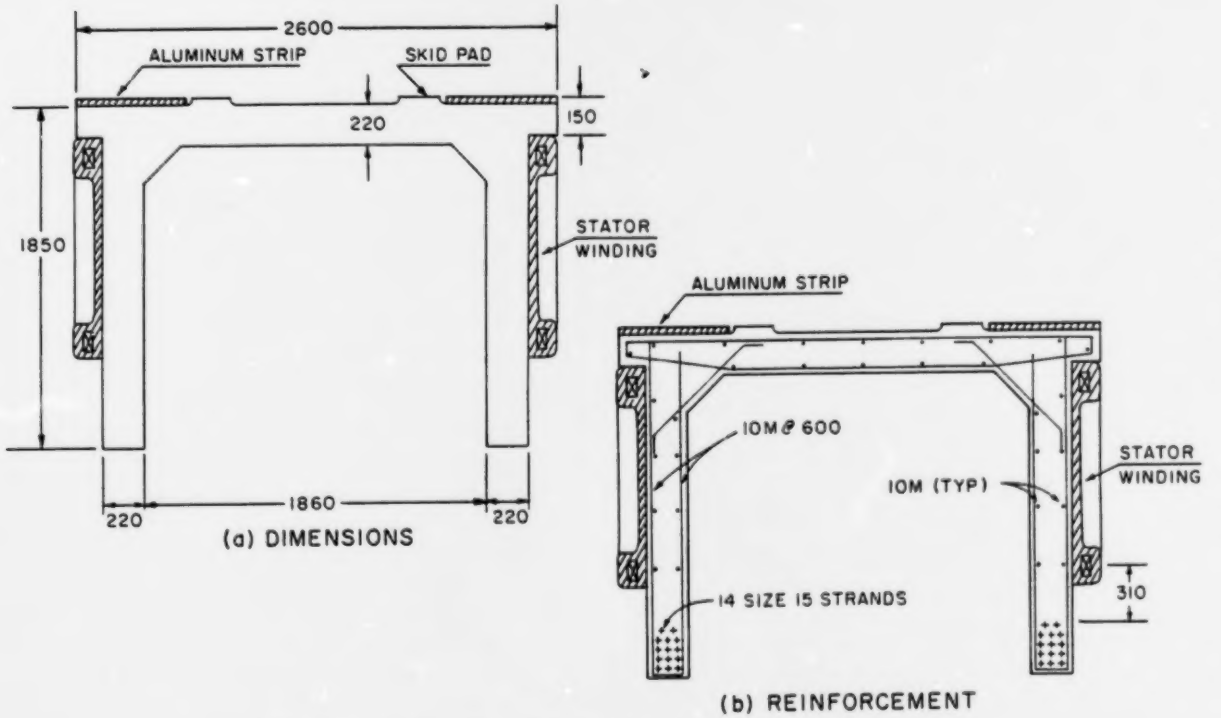

Figure 4.3 Details of channel-section girder

single layer of non-prestressed reinforcement is used in the walls of the box. Two pretensioning strands are located in the centre of the top slab. This girder, which is capable of a span of 25 m, has a fundamental flexural frequency of 4.8 Hz and a mass of 63 t.

Details of the design of the box girder are given in Appendix B.

The box section was found to have a torsional capacity far in excess of that required. Consequently, the possibility of using a channel section which has a lower torsional capacity was examined.

4.2 Design of the Channel-Section Beam

Figure 4.3(a) shows the basic dimensions of the channel section. The overall width is 2,600 mm and the depth is 1,850 mm. The top slab and vertical legs are 220 mm thick. The depth of the channel is greater than that of the box due to the requirement that the longitudinal prestress be 500 mm away from the stator winding. The webs and top slab are thicker than those of the box to accommodate the larger transverse bending moments which occur in the channel section as compared to the box section. As shown in Figure 4.3(b), the pretensioning is concentrated in the bottom of the vertical webs, while two layers of non-prestressed reinforcement are provided in the webs and the top slab. This girder, which is capable of a span of 30 m, has a fundamental flexural frequency of 3.7 Hz and a mass of 93 t.

Details of the design of the channel section girder are given in Appendix C.

The channel section was found to have a more than adequate torsional capacity due to the fact that its shear centre is located above the level of the top slab and the resulting lever arm for lateral loads causing torsional moments is reduced from that in the box section where the shear centre is located close to the centroid of the box section. Locations of the shear centre in the box and channel cross-section are shown in Figure 4.4.

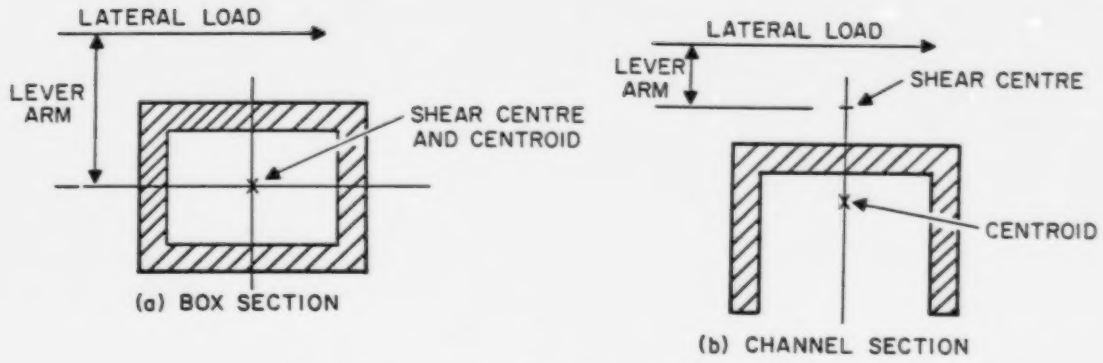

Figure 4.4 Location of shear centre

4.3 Other Sections Considered

Two other sections were considered during the course of the study. These were a box section in which the webs were located closer together than in the section shown in Figure 4.2, and an open section in which the top slab was replaced by transverse diaphragms.

A box section in which the webs were located directly under the skid pads was considered. This section, which would have a lower torsional capacity, would reduce the shear induced in the top slab by the impact loading due to emergency braking. However, a supporting system would have to be provided for the stator windings. This section was discarded in view of the ability of the open channel section to resist the torsional loading and in light of the agreement of two of the principal system developers (Hayes and Eastham) that it would be feasible to replace the wooden skid pads on the vehicle with rubber wheels which would contact the aluminum strip under emergency conditions.

An open section was investigated with a view to reducing the mass of the beam so as to facilitate shipping and field erection. Figure 4.5 shows the cross-sectional dimensions of one of these sections. The girder comprises two concrete I-section beams joined by transverse concrete diaphragms, and is analogous to the channel section with the top slab replaced by the transverse diaphragms. It was proposed that the girders be precast and that the diaphragms be formed in situ. However, subsequent analyses showed that a significant number of

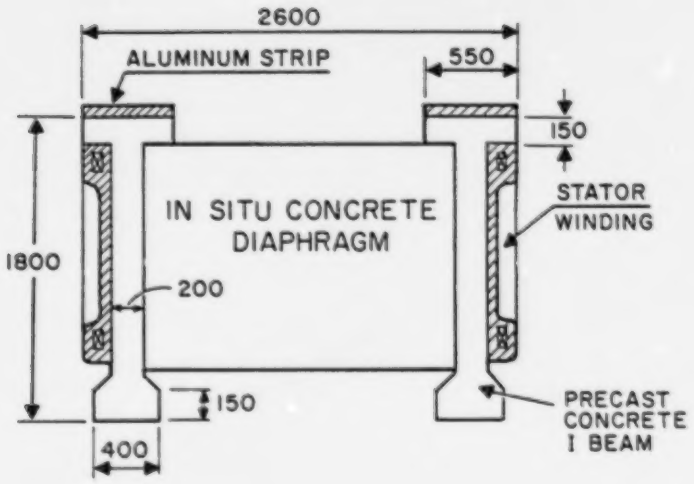

Figure 4.5 Open-section girder with diaphragms

diaphragms would be required to reduce the stress resultants from lateral loads to an acceptable level. Consequently, this design concept was not pursued further.

However, as noted above, the possibility of replacing the wooden skids provided for emergency "landing" of the vehicle on the guideway with fixed wheelsets was investigated. This change would be necessary for beam sections where separate skid pads (see Figures 4.3 and 4.4) could not be provided and the emergency landing gear would have to contact the aluminum strip. This strip could not withstand the impact from skids, but would be capable of withstanding that from a wheeled undercarriage. It is the opinion of the authors that development of this type of emergency landing gear would offer significant advantages regardless of which guideway beam design was used. The corresponding reduction in top slab width and the relocation of the impact loading over the webs of the girders should permit use of a thinner top slab for either the box or channel sections.

4.4 Effects Of Magnetic Reinforcement

Forces will be generated by interaction between the magnetic fields of the vehicle-mounted super-conducting magnets and the steel reinforcing bars in the guideway. The possibility of eliminating these forces by

the use of non-magnetic reinforcement has been studied in Japan.[6] A high-manganese steel, suitable for non-prestressed reinforcement, was developed and has been used in the guideway for the JNR magnetic levitation system.[7] However, the cost of such reinforcement is about three times that of conventional reinforcing steel. Further, the yield strength (400 MPa) of this material is too low to allow its use as prestressed reinforcement. A non-magnetic material suitable for prestressing is the synthetic yarn Kevlar. This material is under study in Britain[8] but is still in the experimental stage. However, it may offer a viable alternative to conventional prestressing steel in the future. Further, this material is not susceptible to corrosion as is conventional prestressing steel.

The influence of the magnetic reinforcement used in the channel section (Figure 4.3) was studied[9] using a finite element discretization to generate the equipotential flux lines for the current in the superconducting coils. Since the simulation is two-dimensional, the discrete vertical reinforcing bars in the web of the section were simulated by a continuous steel sheet of equivalent cross-sectional area. A plot of the equipotential flux lines for the section with only the longitudinal reinforcement present is shown in Figure 4.6. This figure shows that the flux lines are not influenced by the presence of the magnetic reinforcing bars. It was concluded that the force due to the longitudinal bars is negligible and that the force due to the vertical bars is small and can be tolerated. Consequently, the use of conventional steel reinforcements for the guideway appears reasonable.

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6. Okuda, T. "Possibility to Reinforce Concrete with Non-magnetic High Manganese Steel Bar." Quarterly Reports, Railway Technical Research Institute, Vol. 22, No. 1, Japan National Railways, March 1981, pp. 38-39.

7. Fujii, H. "Present Situation Concerning JNR's Magnetic Levitation System." Civil Engineering in Japan, Vol. 22, Society of Civil Engineers, 1983, pp. 127-136.

8. Burgoyne, C.J., and J.J. Chambers. "Prestressing with Parofil Tendons." Concrete, October 1985, pp. 12-15.

9. Eastham, A.R., and G. Mizia. Analysis of the Lateral Force on a Canadian Maglev Guideway. Research Report, Department of Electrical Engineering, Queen's University, Kingston, Ontario, May 1986, 6 pages.

Further, since the effect of longitudinal bars is negligible, harping of pretensioning strands located in the webs of the girder could be utilized to control stress levels at the end of the girders.

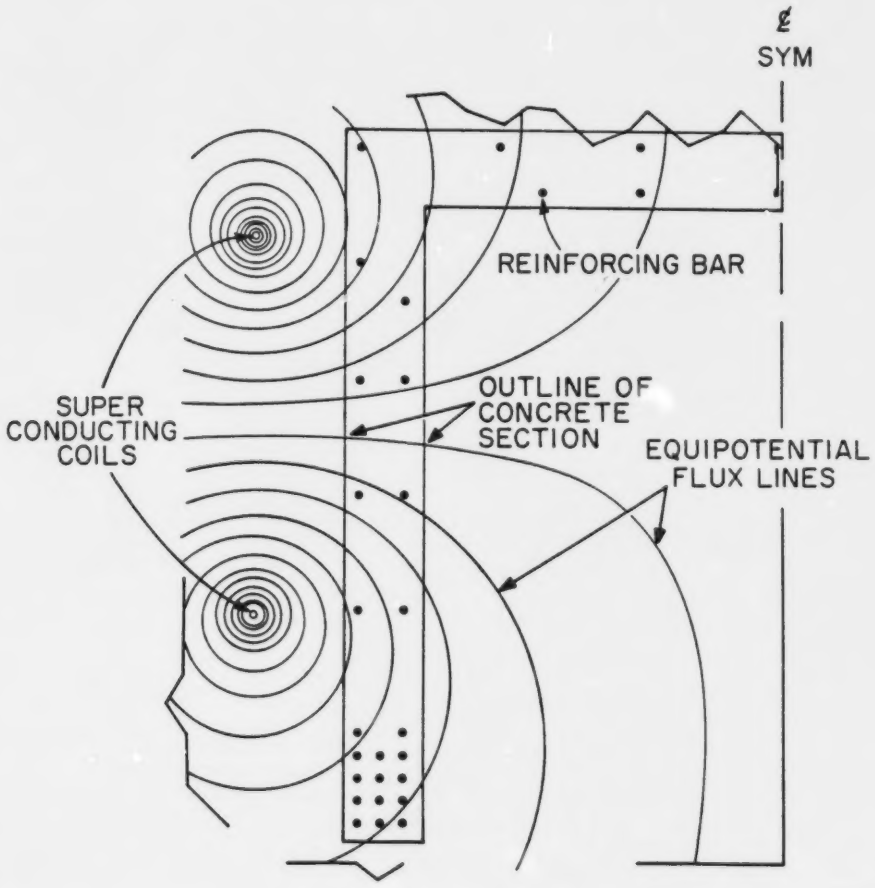


Figure 4.6 Equipotential flux lines

To reduce the drag effect due to eddy currents induced in closed loops of reinforcing steel in the guideway, the reinforcing bars will have to be epoxy-coated to provide effective electrical insulation. Epoxy-coated reinforcement is commonly used in bridge construction in North America to combat corrosion. The additional cost of epoxy-coating the reinforcement is discussed in Section 5.

4.5 Tolerances

A guideway constructed to tolerances required for highway bridge construction should be suitable for the Maglev vehicle. Typical tolerances, relating to the parameters identified in Figure 4.7, for bridge girders[10] are given in Table 4.4. Camber values are taken from a publication of the PCI Committee on Tolerances.[11] Past experience has shown that these tolerances can readily be met by North American precast concrete manufacturers.

TABLE 4.4

TYPICAL CONSTRUCTION TOLERANCES
FOR CONCRETE BRIDGE GIRDERS

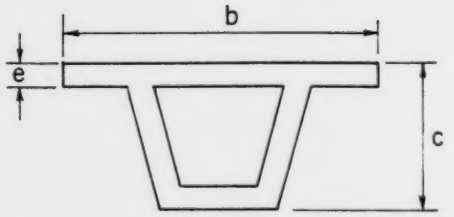
Parameter	Tolerance (mm)
Width, b	±3
Depth, c	+8,* +12**
Flange thickness, e	±5
Sweep, j	±3/3 m, +15 max.
Camber, k	+6/3 m, +19 max.

* depth ≤ 600 mm
** depth > 600 mm

Design calculations shown in Appendix B for the box section indicate that the long-time camber is in excess of the 19 mm maximum given in Table 4.4. This camber could be reduced by using harped tendons rather than straight as assumed in the computations. Harped tendons are possible if the effects of magnetic reinforcing steel can be tolerated. However, camber growth should be studied in detail in the course of the final design of the guideway.

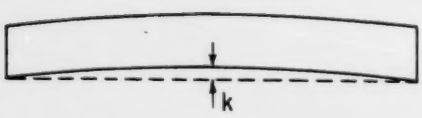
10. "Construction Specifications for Prestressed Concrete Members." Ontario Provincial Standards Specifications, Vol. 1, Construction, December 1983.

11. PCI Committee on Tolerances. "Tolerances for Precast and Prestressed Concrete." PCI Journal, Vol. 80, No. 1, January/February 1985, pp. 26-112.



CROSS SECTION

PLAN



ELEVATION

Figure 4.7 Definition of tolerance parameters

5. DISCUSSION OF RESULTS

Cost estimates were initially obtained from two companies, Schokbeton Quebec Inc. of St. Eustache, Québec, and Pre-Con of Brampton, Ontario. The former organization prepared a detailed quotation, reproduced here as Appendix D, under a small subcontract. Pre-Con declined the offer of a similar subcontract, and prepared its less well documented quotation at no direct cost to the project. Subsequently, as a result of the very substantial variance between the two quotes, an offer by Supercrete of Richmond, B.C., to also prepare an estimate (on a no-cost basis) for the two revised beam designs was accepted. As discussed above, Pre-Con and Schokbeton also gave estimates for the precast trapezoidal trough of the original design. These cost estimates which are based on f.o.b. plant, and do not include federal and provincial sales tax, are given in Table 5.1.

TABLE 5.1

COMPARISON OF BASIC UNIT COST ESTIMATES
($1986/linear metre)

	Cost			Cost Ratio	
Section	Pre-Con (1)	Schokbeton (2)	Supercrete (3)	(1)/(2)	(3)/(2)
Box	1,773	566	730	3.13	1.29
Channel	1,322	521	620	2.54	1.19
Trapezoidal trough	1,436	442	-	3.25	-

The estimates from Schokbeton are based on a ten-year production program while those of Supercrete assume a five-year production program. Additional data provided by Schokbeton indicate that reduction from a ten- to a six-year production program would result in a cost increase of $11/m for the box or the channel section and $5/m for the trapezoidal trough. Cost does not appear to be particularly sensitive to the length of the production period.

Table 5.1 also presents the ratio of the Pre-Con and Supercrete estimates relative to the Schokbeton estimate.

The estimates from Pre-Con are in excess of those from Schokbeton by factors of 3.13 for the box section, 2.54 for the channel section, and 3.25 for the trapezoidal trough section, while those from Supercrete are in excess by factors of 1.29 for the box section and 1.19 for the channel section. The latter variance is well within the "typical" range for competitive bids on jobs of this nature. The source of the unusually large variance associated with the Pre-Con quotation could not be established, despite repeated verbal, written and direct contact inquiries by project staff. The reasonable agreement between the estimates from Supercrete and Schokbeton indicate that the Pre-Con estimates are unrealistically high. (This variance clearly demonstrates the importance of obtaining competitive bids for large jobs.)

The cost estimates for the box and channel sections are compared in Table 5.2. The cost estimates for the box section are consistently above those for the channel section by factors of 1.34, 1.09 and 1.18 for Pre-Con, Schokbeton and Supercrete, respectively. This suggests that a channel section would be (conservatively) 10 to 20 percent less costly than a box section.

TABLE 5.2

COMPARISON OF COST RATIOS FOR BOX AND CHANNEL SECTIONS

	Pre-Con	Schokbeton	Supercrete
Box/Channel Cost Ratio	1.34	1.09	1.18

Table 5.3 compares the estimated costs of the box and channel section beams with those for the completed trapezoidal trough beam. It is clear that the channel section beam offers substantial economies over both the original design and the box-section beam. The longer span (30 m versus 25 m) of the channel section would reduce the number of beams to be produced, shipped and field-erected by 7,893. More importantly, it would also reduce the number of columns, crossheads and footings by half the above number -- 3,946. This would correspond to a savings of about $36 million ($1986).

TABLE 5.3

COMPARISON OF ORIGINAL AND REVISED GUIDEWAY BEAM COSTS
($1986/linear metre)

	Original			Revised					
				Box			Channel		
	Escalated from $1978 (1)	Pre-Con (2)	Schokbeton (3)	Pre-Con (4)	Schokbeton (5)	Supercrete (6)	Pre-Con (7)	Schokbeton (8)	Supercrete (9)
$/lin. metre	$873	$1,776	$782	$1,773	$566	$730	$1,322	$521	$620
Beam span	25	25	25	25	25	25	30	30	30
No. of beams required (592 km x 2)	47,360	47,360	47,360	47,360	47,360	47,360	39,466	39,466	39,466
Cost per beam	$21,825	$44,400	$19,550	$44,325	$14,150	$18,250	$39,660	$15,630	$18,600
Total beam cost ($1986 millions)	$1,033.6	$2,102.8	$925.9	$2,099.2	$670.1	$864.3	$1,565.2	$616.8	$734.1

Table 5.4 summarizes the cost savings (increases) for the two revised sections relative to both the escalated cost of the original design and to the corresponding cost as estimated in the current study.

It is clear that the channel section would offer significant cost savings relative to either the original trapezoidal trough/top slab design or the box-section beam.

If the Schokbeton price levels for both the original and revised designs could be achieved, the savings with the channel section would amount to about $346 million ($1986), or about 37 percent of the re-estimated cost for the original "Alternatives to Air" beam. With the Supercrete price levels, the savings would be about $229 million ($1986), or just under 25 percent of the cost for the original beam design.

The Pre-Con price estimate for the original beam design is more than twice the escalated "Alternatives to Air" cost, and no adequate

explanation for this anomalously high value has been established. In relative terms, however, even the Pre-Con estimates confirm the superiority of the channel-section design, which would save over $600 million ($1986) relative to the Pre-Con estimate for the original design.

TABLE 5.4

COST SAVINGS (INCREASES) WITH REVISED BEAM DESIGNS
($1986 millions)

	Savings (Increase) Relative to Escalated "Alternatives to Air" Cost				Savings (Increase) Relative to Re-estimated Cost for Original Design							
					Schokbeton Re-estimate				Pre-Con Re-estimate			
Design	Beam ($)	Other ($)	Beam (%)	Other (%)	Beam ($)	Other ($)	Beam (%)	Other (%)	Beam ($)	Other ($)	Beam ($)	Other ($)
Box:												
.Schokbeton	364	-	35	-	256	-	28	-	1,433	-	68	-
.Supercrete	169.3	-	16	-	62	-	7	-	1,239	-	59	-
.Pre-Con	(1,066)	-	(103)	-	(1,173)	-	(363)	-	4	-	0.2	-
Channel:												
.Schokbeton	417	36	40	4	309	36	33	4	1,486	36	71	1.5
.Supercrete	300	36	29	4	192	36	21	4	1,369	36	65	1.5
.Pre-Con	(532)	36	(51)	4	(639)	36	(69)	4	538	36	26	1.5

6. CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of this study, the following conclusions are offered:

- Both the box-section and channel-section beam designs would meet all technical and regulatory requirements for this type of installation.
- Conventional (magnetic) steel reinforcements can be used in construction of the guideway beams without causing adverse effects on vehicle operations or on guideway life expectancy.
- The channel section would be less costly to fabricate than the box section, by about 10 to 25 percent.
- The channel section would offer significant cost savings over the original trapezoidal trough/top slab design. Relative to the corresponding revised costs for the original design, the channel section would permit savings in the range of 15 to 35 percent.
- The Schokbeton price levels for the channel section, if achieved, would correspond to a savings of about 42 percent (including the savings from fewer support structures) relative to the escalated "Alternatives to Air" beam cost.
- The use of epoxy-coated reinforcing bars will increase the cost of the guideway girders by 10 to 20 percent.

Based on the foregoing discussion and conclusions, the following recommendations are offered:

- An analagous design review and recosting of the guideway support structure should be undertaken, together with investigation of transportation and site erection costs, with the objective of establishing a "commercial price" range for the complete guideway in place.

- All participating industrial subcontractors shoud be retained on a remunerated basis. The work performed by Schockbeton under a remunerated subcontract was far more satisfactory than that of Pre-Con which provided its estimate on a "no-cost" basis.12

- If the apparent savings established in this phase are substantiated by the investigations suggested above, funding of the second phase of Project 441 should be given serious consideration, as should the parallel investigation of possible guideway configurations.13

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12. This arrangement was at Pre-Con's request.

13. As proposed in CIGGT Project PRO-538.

APPENDIX A

MAGLEV GUIDEWAY STRUCTURAL DESIGN

This appendix was extracted directly from CIGGT Report No. 80-4, "Alternatives to Air: A Feasible Concept for the Toronto-Ottawa-Montréal Corridor." Original numbering has been retained.

APPENDIX 3.B

MAGLEV GUIDEWAY STRUCTURAL DESIGN

The following structural calculations outline the iterative process of structural design to establish the physical dimensions of the guideway beam and beam support structure used in the costing of the Maglev system concept. The calculations, as such, are not developed to the level required for construction purposes, but afford a degree of accuracy sufficient to authenticate the design and fulfill the requirements of the costing methodology.

The column, crosshead and beam have been examined to the level of construction feasibility, the footing concept adopted being conservatively sized to conform to the nominal bearing capacity utilized. The overall system concept is shown to be feasible and well within the realms of present-day precast concrete construction technology. As emphasized earlier in the text, the advances in such technology by the time a project of this magnitude is implemented are likely to have a major impact both on overall size of system components and total system concept.

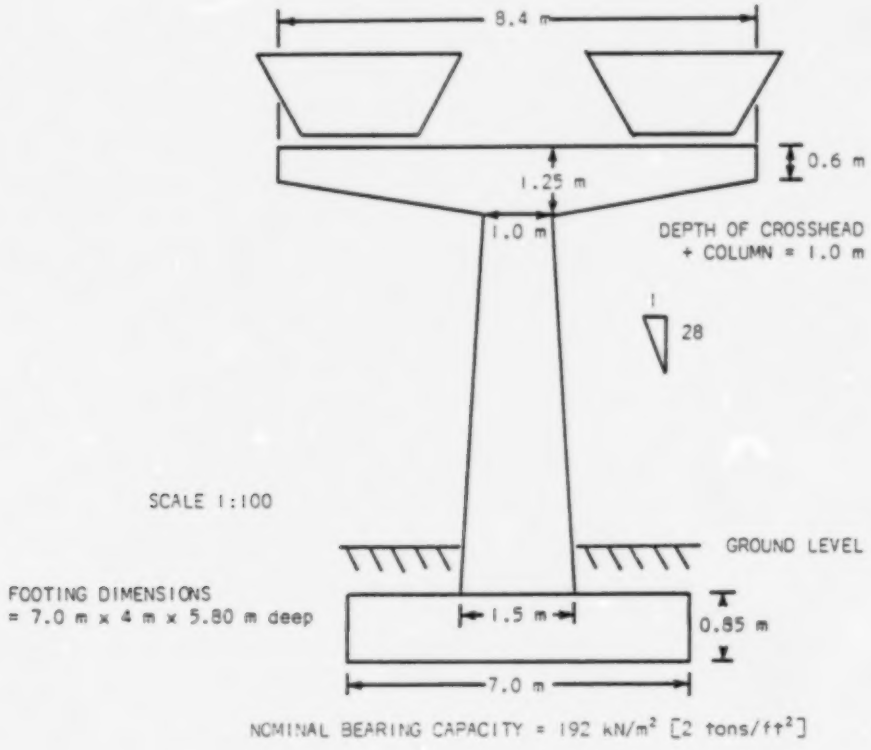

Figure 3.B.1 Support system dimensions

1. Check guideway beam design

1(a). Check compatibility of guideway design with revised loadings (10 vs 8 magnets)

2. Design of double-beam column

3. Footings

4. Walkway

5. Single guideway concept

6. Costing of structures

7. Special structures

8. Summary of costs

3.B.1 ANALYSIS OF GUIDEWAY BEAM DESIGN

The basic Maglev beam design developed by Campbell* has been modified slightly to incorporate a thicker top slab (152.4 mm vs 101.6 mm) to accommodate the slots for the LSM windings and null-flex loops, as shown in Figure 3.B.3. This alteration, plus the increase in vehicle weight arising from the use of economy (145 passenger) seating, has necessitated the following check on the beam design.

STEP 1: Lower section of beam will be cast with 0.5" ∅ pretension strands (A_{sp} = 0.1438 in^2/strand) at 2" spacing centre-to-centre.† This channel section has been modelled as shown in Figure 3.B.2 to simplify the design calculations.

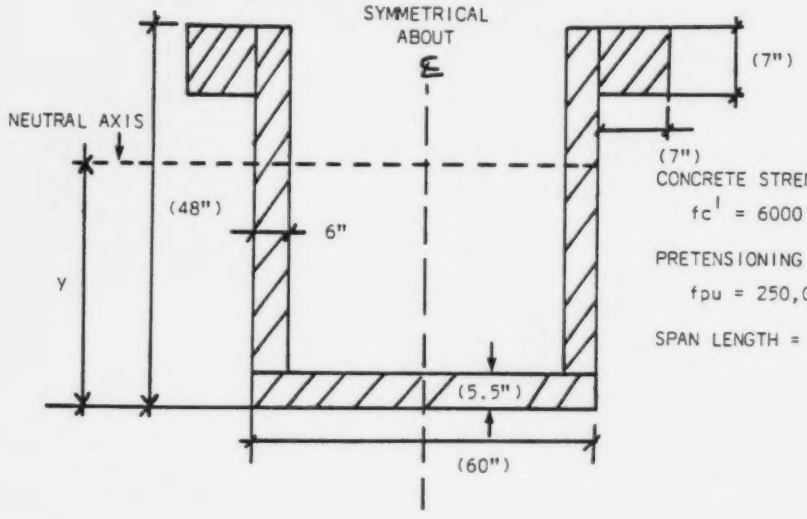

Figure 3.B.2

Model channel section of beam

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* Dr. T.I. Campbell, Dept of Civil Engineering, Queen's University.

† 38.10 mm c/c spacing cannot be used, since this would violate Code CAN3-A23.3-M77

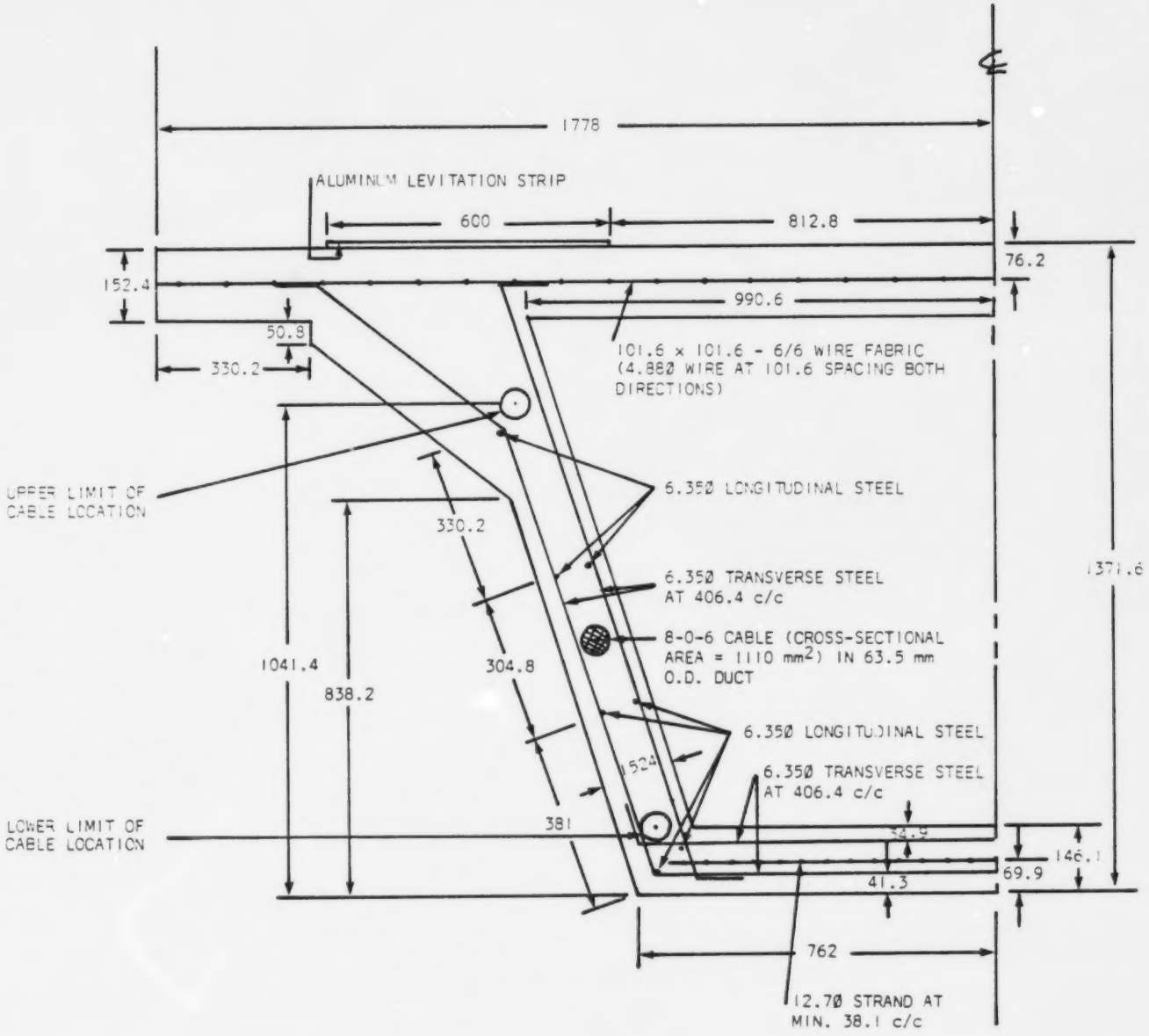

Figure 3.B.3 Cross-section, Maglev guideway beam (all dimensions in mm - beam is symmetrical about centre line)

3.B.1.1 Dead Load of Channel Section (W'_{dead})

$$\text{Area} = 2\,[(7)(7) + (6)(48-5.5)] + (60)(5.5)$$
$$= 6.51\ ft^2\ (0.605/m^2)$$

$$\gamma\ \text{Concrete} = 150\ lb/ft^3\ \ (2{,}403\ kg/m^3)$$
$$\therefore\ w'_{dead} = (150\ lb/ft^3)(6.51\ ft^2) = 977\ lb/ft$$

The maximum moment due to the dead load =

$$Mg' = \frac{w'_{dead} \times \ell^2}{8} = \frac{(977\ lb/ft)(82^2)}{8} = 821{,}000\ lb\text{-}ft$$

3.B.1.2 Calculation of Pretension Force

Pretension force before losses (P_i) = $A_{st} \cdot \sigma_{allow}$

Now, $A_{st} = \frac{\text{width}}{\text{spacing}} \cdot$ area of strand

$$= \frac{60"}{2"} \cdot .1438 = 4.31/in^2\ \ (2.783 \times 10^{-3}\ m^2)$$

and $\sigma_{allow} = 0.70^{*}\ f_{pa}$
$= (0.70)(250{,}000\ psi)$
$= 175{,}000\ psi$

$P_i = (175{,}000\ psi)(4.31\ in^2)$
$= 7.54 \times 10^5\ lb$

3.B.1.3 Calculation of Moment of Inertia of Channel Section

3.B.1.3.1 Locate neutral axis (NA)

Transform area of pretension steel to equivalent area of concrete:

$$\text{modular ratio } \frac{E_{st}}{E_c} = \frac{2.9 \times 10^7\ psi}{5.7 \times 10^4 \sqrt{f'_c}} = \frac{2.9 \times 10^7\ psi}{5.7 \times 10^4 \sqrt{6000}} = 6.57,$$

which is rounded to 7.

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* Winter, G, Nilson, A.H., *Design of Concrete Structures*, McGraw-Hill Book Co., 1972.

The equivalent additional area of concrete is $(n-1)\ A_{sp}$

$$= 6(4.31)$$

$$= 25.9\ in^2$$

and the equivalent additional width to the bottom flange is given by

$$\frac{\text{area}}{\text{thickness}} = \frac{25.9}{5.5} = 4.7 \text{ inches.}$$

Now, the moment of the area above the neutral axis must equal the moment of the area below the neutral axis, so that

$$M_{AA} = M_{AB}$$

Letting y equal the height of the neutral axis gives

$$M_{AA} = (2)(7)(7)(48-y-3.5) + (2)(6)\frac{(48-y)^2}{2}$$

$$= -98\ y + 4{,}361 + 13{,}824 - 576\ y + 6\ y^2$$

$$\underline{M_{AA} = 6\ y^2 - 674\ y + 18{,}185}$$

and

$$M_{AB} = (2)(6)\frac{(y-5.5)^2}{2} + (5.5)(64.7)(y-2.75)$$

$$= 6\ y^2 - 66\ y + 181.5 + 355.9\ y - 978.6$$

$$\underline{M_{AB} = 6\ y^2 + 289.9\ y - 797.1}$$

Since $M_{AA} = M_{AB}$,

$$6\ y^2 - 674\ y + 18{,}185 = 6\ y^2 + 289.9\ y - 797.1$$

$$963.9\ y = 18{,}982.1$$

$$y = 19.7 \text{ inches}$$

3.B.1.3.2 Calculate Moment of Inertia
(uncracked section)

$$I = (2)\frac{(7)(7)^3}{12} + (2)(7)(7)[48-19.7-3.5]^2 + \frac{(2)(6)(48-5.5)^3}{12}$$

$$+ (2)(6)(48-5.5)[48-19.7-(\frac{48-5.5}{2})]^2 + \frac{(64.7)(5.5)^3}{12}$$

$$+ (64.7)(5.5)(19.7-2.75)^2$$

$$= 2.66 \times 10^5 \text{ in}^4 = 12.8 \text{ ft}^4$$

3.B.1.3.3 Check Outer Fibre Stresses on Transfer of Pretension Force

Top fibre stress (compressive stresses ≡ negative)

$$f_{1,p_i} = -\frac{P_i}{A} + \frac{P_i e c_1}{1} - \frac{My\ c_1}{I}$$

where: f_{1,p_i} = the stress in the top fibre before pretension losses occur

A = area of cross-section

P_i = initial pretension force (before losses)

e = eccentricity of pretension force from the neutral axis

c_1 = the distance from the neutral axis to the top of the section

I = the moment of interia of the section

My = dead load moment

$$\therefore f_{1,p_i} = -\frac{7.54 \times 10^5}{9.38 \times 10^2} + \frac{(7.54 \times 10^5)(19.7 - 2.75)(48 - 19.7)}{2.66 \times 10^5}$$

$$- \frac{(9.85 \times 10^6)(48 - 19.7)}{2.66 \times 10^5}$$

$$f_{1,p_i} = -4.92 \times 10^2 \text{ psi}$$

Immediately after transfer, the allowable compressive stress

$$f_{1,p_i,allow} = 0.60\ f'_{c_1}$$

where f'_{c_1} = compressive strength of the concrete at the time of the initial prestress.

Assuming the concrete has reached full strength,

$$f_{1,p_i,allow} = (0.60)(6000) = 3,600 \text{ psi}$$

so that

$$F.S. = \frac{3,600}{492} = 7.3, \text{ which is acceptable.}$$

Therefore, the top of the web is capable of sustaining the pretension force.

Bottom fibre stress:

$$f_{2,p_i} = -\frac{P_i}{A} - \frac{P_i e c_2}{I} + \frac{My\ c_2}{I}$$

where: f_{2,p_i} = the stress in the bottom fibre before pretension losses occur

c_2 = the distance from the neutral axis to the bottom of the section

$$f_{2,p_i} = -\frac{7.54 \times 10^5}{9.38 \times 10^2} - \frac{(7.54 \times 10^5)(19.7-2.75)(19.7)}{2.66 \times 10^5} + \frac{(9.85 \times 10^6)(19.7)}{2.66 \times 10^5}$$

$$= -1,021 \text{ psi}$$

and $F.S. = \frac{3,600}{1,021} = 3.5$, which is again acceptable.

Therefore, the bottom flange is capable of sustaining the pretension force.

3.B.1.4 Effect of Top Slab

The next step is to consider the performance of the beam after the top slab has been cast. The static loads imposed on the completed beam include the guidance rails and supports, the levitation strips, the LSM windings and null-flux loops, the dead load of the beam itself, and the dead load of a stopped, fully-loaded 145-seat vehicle. To simplify the analysis, the average density of the LSM windings and null-flux loops and their surrounding lightweight plastic insulation was taken to be similar to that of concrete. This allowed the top slab to be treated as a solid block of concrete, even though the windings and insulation are installed in slots cast into the upper surface of the slab.

3.B.1.4.1 Guidance Rails and Supports

Each beam carries eight supports and two guidance rails; for simplicity, it has been assumed that the support posts are constructed of the same material as the

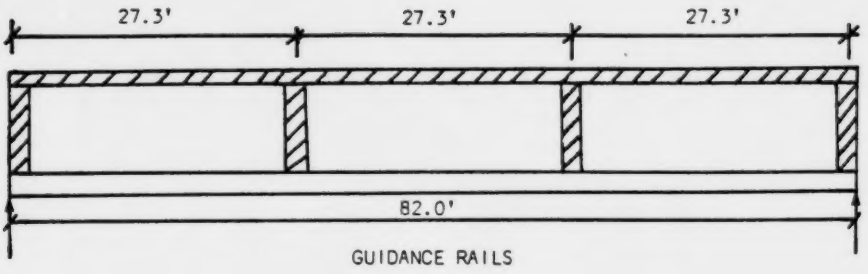


Figure 3.B.4 Guidance rails

rails--W12 x 53 I-beam--for a total positive guidance load of 11,754 lbs (5,343 kg), or 143.34 lb/ft.

This load can be treated as three point loads, one of which is at a column and is thus not an effective load. The effective point load, P, is thus 11,754 lb ÷ 3 = 3,918 lb.

3.B.1.4.2 Levitation Strip Load

The load from the levitation strips is uniformly distributed over the length of the span. This load is equal to:

$$\{2\ (.01m \times .60m \times 25m)\ (2{,}768\ kg/m^3)\ (2.205\ lb/kg)\} \div 82 = 22.3\ lb/ft$$

3.B.1.4.3 Vehicle Load

The load imposed by a fully-loaded 145-passenger vehicle is

$$\omega_{live} = \frac{\text{Vehicle mass}}{\text{Vehicle length}} = \frac{34{,}000\ kg}{32\ m} = 1{,}063\ kg/m = 714\ lb/ft,$$

while,

T_{live} = live load dynamic factor = 1.7[†]

The effective live load is thus: $\omega^*_{live} = \omega_{live} \cdot T_{live}$

$= 714\ lb/ft \cdot 1.7$

$= 1{,}214\ lb/ft$

[†] Wormley, D.N.; Smith, C.C.; Gilchrist, A.J.; "Multispan Elevated Guideway Design for Passenger Transport Vehicles" U.S. Dept. of Transport Report No. FRA-DR & D-75-43 I.

3.B.1.4.4 Dead Load of Entire Beam (with top slab)

The cross-sectional area is approximately given by:

$$A = 2\left[(70)\ (6) + \frac{1}{2}\ (12)\ (10) + (45)\ (6) + (30)\ (5\tfrac{1}{2})\right]$$

Top Slab Web Supports Web Bottom Flange

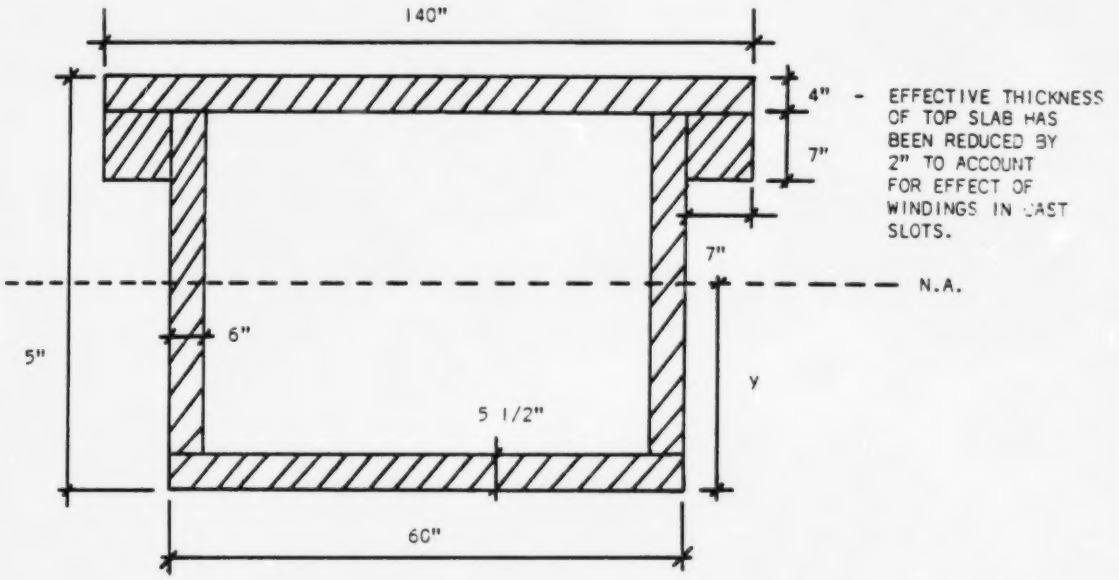
$$= 1.83 \times 10^3\ in^2$$

$$\omega_{dead} = (1.83 \times 10^3\ in^2)\left(\frac{150\ lb}{ft^3}\right)\left(\frac{1\ ft^2}{144\ in^2}\right)$$

$$= 1.91 \times 10^3\ lb/ft$$

3.B.1.5 Moment of Inertia Calculation

For the purposes of analysis, the cross-section configuration has been modelled as shown below.



3.B.1.5.1

The area of reinforcing steel in the top flange is given by

$$A_{steel} = \frac{\text{width}}{\text{spacing}} \times \text{area of strand}$$

$$= \left(\frac{140}{4}\right) \times \left(\frac{\pi\,(.192)^2}{4}\right) = 1.01 \text{ in}^2$$

and the equivalent area of concrete by

$$A_{equiv.} = (n - 1)\, A_{steel}$$

$$= (7-1)\,(1.01)$$

$$= 6.06 \text{ in}^2$$

The additional top-flange width is thus $\frac{6.06}{4} = 1.52$ in, and the additional bottom flange width, as previously calculated, is 4.7 in.

3.B.1.5.2 Location of Neutral Axis

Again, the moments of the areas above and below the neutral axis must be equal:

$$M_{AA} = M_{AB}$$

Allowing y to equal the height of the neutral axis,

$$M_{AA} = (4)\,(141.52)\,(52 - y - 2) + (2)\,(7)\,(7)\,(52 - y - 7.5) + (2)\,(6)\,\left(\frac{52 - y - 4}{2}\right)$$

$$= 6y^2 - 1{,}240y + 46{,}489$$

and

$$M_{AB} = (2)\,(6)\,\frac{(y - 5.5)^2}{2} + (64.7)\,(5.5)\,(y - 2.75)$$

$$= 6y^2 + 289.9y - 797$$

So that $1{,}529.9y = 47{,}286$

and $y = 30.91$ inches

3.B.1.5.3 Calculation of Moment of Inertia

The moment of inertia, I is given by

$$I = (4)(141.52)(52 - 30.9 - 2)^2 + (2)(7)(7)(52 - 30.9 - 7.5)^2$$

$$+ \frac{(2)(6)(52 - 4 - 5.5)^3}{12} + (2)(6)(52 - 4 - 5.5)$$

$$\left[30.9 - \left(\frac{52 - 4 - 5.5}{2}\right) - 5.5\right]^2$$

$$+ (64.7)(5.5)(30.9 - 2.75)^2$$

$$I = 28.6 \text{ ft}^4$$

3.B.1.6 Effect of Post-Tensioning

When the cables running through the webs of the guideway are post-tensioned, the stress level in the pre-tensioning wires will change.

Strength Analysis

At mid-span, the top A fibre stress will be

$$f_1 = -\frac{P_e}{A} + \frac{P_e e_1 c_1}{I} - \frac{p^1}{A} + \frac{p^1 e_2 c_1}{I} - \frac{(M_D + M_L)\, C_1}{I}$$

where:
- P_e = pretension force remaining after post tensioning
- A = area of cross section
- e_1 = eccentricity of pretension force with neutral axis
- c_1 = distance from the neutral axis to the top fiber of the beam
- I = the moment of inertia of the entire cross section
- p^1 = the post tension force
- e_2 = the eccentricity of the post tension force with the neutral axis
- $M_D + M_L$ = the moment generated at mid span due to the dead and live loads

Assuming a 20% loss in pretension force after transfer, the pretension force before post tensioning, becomes 1.40×10^5 psi

The strain introduced at the level of the pretension steel by the post-tensioning is given by

$$\varepsilon = \frac{\sigma}{Ec} = \frac{1}{Ec}\left[-\frac{P^1}{A} - \frac{P^1 e_2 e_1}{I}\right]$$

where, a compressive stress is taken to be negative, and

$$P^1 = (2)\ (250{,}000)\ (1.72)\ (0.70) = 6.02 \times 10^5 \text{ lb.}$$

$$A = 1.83 \times 10^3 \text{ in}^2$$

$$e_2 = 30.9 - 5.5 = 25.4 \text{ in}$$

e_1 = distance from N.A. to centroid of pretension steel

$$= 30.9 - 2.75 = 28.15 \text{ in}$$

$$I = 5.92 \times 10^5 \text{ in}^4$$

$$Ec = 57{,}000\sqrt{f'_c} = 4.42 \times 10^6 \text{ psi}$$

$$\varepsilon = \frac{1}{4.42 \times 10^6}\left[-\frac{6.02 \times 10^5}{1.83 \times 10^3} - \frac{(6.02 \times 10^5)\ (25.4)\ (28.15)}{5.92 \times 10^5}\right]$$

$$= -2.39 \times 10^{-4} \text{ in/in}$$

$$\Delta f \text{ pretension steel} = E_{st} \cdot \varepsilon = (29 \times 10^6)\ (-2.39 \times 10^{-4})$$

$$= -6.93 \times 10^3 \text{ psi}$$

Consequently, the estimated pretension force *after posttensioning* is

$$P_e = [1.40 \times 10^5 - 6.93 \times 10^3]\ (4.31) = 5.74 \times 10^5 \text{ lb}$$

3.B.1.7 Calculation of Moments due to the Loads

3.B.1.7.1 Midspan Moments

The dead-load moments result from the positive guidance system, the levitation strips, and the dead weight of the guideway beam itself. For the positive guidance system, the mid span moment is given by

$$M_s = Pa$$

In this case, $a = \frac{\ell}{3}$, where P = 3,918 lb and ℓ = 82 ft

Hence, the mid-span moment equals

$$\frac{(3{,}918 \text{ lb})\ (82 \text{ ft})}{3}$$

$$= 1.07 \times 10^5 \text{ lb-ft}$$

For the levitation strips, the mid span moment is given by

$$\frac{wl^2}{8}, \quad \text{where} \quad w = 22.3 \text{ lb/ft}$$

which equals $$\frac{(22.3 \text{ lb/ft}) \ (82.0 \text{ ft})^2}{8}$$

$$= 1.87 \times 10^4 \text{ lb-ft}$$

For the dead weight of the guideway, the midspan moment is again given by

$$\frac{wl^2}{8}, \quad \text{where} \quad w = 1.91 \times 10^3 \text{ lb/ft}$$

which equals $$\frac{(1.91 \times 10^3 \text{ lb/ft}) \ (82.0 \text{ ft})^2}{8}$$

$$= 1.61 \times 10^6 \text{ lb-ft}$$

and M_D, the total dead load moment at midspan,

$$= 1.07 \times 10^5 + 1.87 \times 10^4 + 1.61 \times 10^6 \text{ lb-ft}$$
$$= 1.73 \times 10^6 \text{ lb-ft}$$

The live-load mid-span moment is given by

$$\frac{wl^2}{8}, \quad \text{where} \quad w = \omega^*_{live} = 1{,}214 \text{ lb/ft}$$

which amounts to $$\frac{(1.214 \times 10^3 \text{ lb/ft}) \ (82.0 \text{ ft})^2}{8}$$

$$= 1.02 \times 10^6 \text{ lb-ft}$$

The total moment due to loads M_D and $M_L = 2.75 \times 10^6$ lb-ft

$$\therefore f_1 = -\frac{P_e}{A} + \frac{P_e e_1 C_1}{I} - \frac{P^1}{A} + \frac{P^1 e_2 C_1}{I} - \frac{(M_D + M_L)\ C_1}{I}$$

and $Pe = 5.74 \times 10^5$ lb

$P^1 = 6.02 \times 10^5$ lb

$A = 1.83 \times 10^3$ in^2

$I = 5.92 \times 10^5$ in^4

$e_1 = 28.15$ in

$$e_2 = 25.4 \text{ in}$$

$$C_1 = 21.10 \text{ in}$$

$$M_D + M_L = 2.75 \times 10^6 \text{ lb-ft} = 3.3 \times 10^7 \text{ lb-in}$$

$$\text{so } f_1 = -\frac{5.74 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} + \frac{(5.74 \times 10^5 \text{ lb})\ (28.15 \text{ in})\ (21.1 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{6.02 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} + \frac{(6.02 \times 10^5 \text{ lb})\ (25.4 \text{ in})\ (21.1 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{(3.3 \times 10^7 \text{ lb-in})\ (21.10 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$= -697.9 \text{ psi}$$

The allowable compressive stress is $0.45\ f'_c = 0.45\ (6{,}000) - 2{,}700$ psi

$\therefore$ the Factor of Safety is $\frac{2{,}700}{697.9} = 3.9$, which is entirely satisfactory.

The bottom fibre stress, f_2, is given by

$$f_2 = -\frac{P_e}{A} - \frac{P_e e_1 C_2}{I} - \frac{P^1}{A} - \frac{P^1 e_2 C_2}{I} + \frac{(M_D + M_L)\ C_2}{I}$$

$$= -\frac{5.74 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} - \frac{(5.74 \times 10^5 \text{ lb})\ (28.15 \text{ in})\ (30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{6.02 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} - \frac{(6.02 \times 10^5 \text{ lb})\ (25.4 \text{ in})\ (30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$+ \frac{(3.3 \times 10^7 \text{ lb-in})\ (30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$= -561.7 \text{ psi, which is again in compression, so that the safety factor is: } FS = \frac{2{,}700}{561.7} = 4.8$$

3.B.1.7.2 Quarter-Span Moments

To check the stress at ℓ/4 (20.5') from the supports, it will be necessary to find an expression for the eccentricity of the post-tensioning cable as a function of distance from the end of the span.

Assume the cable profile is parabolic, $e_2 = ax^2 + bx + c$, where x is the distance from the span-end in feet.

The boundary conditions (positions above the neutral axis are negative) are

(1) $e_{2/x = 0} = -41" + 30.9" = -10.1"$

(2) $e_{2/x = \ell/2 = 41'} = 30.9" - 5.5" = 25.4"$

(3) $e_{2/x = \ell = 82'} = -10.1"$

from (1) $e_{2/x = 0} = -10.1 = C \qquad C = -10.1$

(2) $e_{2/x = \ell/2} = 25.4 = a(41)^2 + b(41) - 10.1$

or $1,681a + 41b = 35.5 \quad \ldots\ldots (4)$

from (3) $e_{2/x = \ell} = -10.1 = a(82)^2 + b(82) - 10.1$

or $6,724a + 82b = 0 \quad \ldots\ldots (5)$

from (5) $-[2 \times (4)]$

$$\begin{array}{r} 6,724a + 82b = 0 \\ -(3,362a + 82b = 71.0) \\ \hline 3,362a = -71.0 \end{array}$$

$a = -0.0211 \quad \ldots\ldots\ldots (6)$

from (6) and (4)

$1,681\ (-0.0211) + 41b = 35.5$

$41b = 35.5 + 1,681\ (0.0211)$

$b = 1.73$

$e_2 = -0.0211x^2 + 1.73x - 10.1$ where e_2 is in inches and x is in feet.

Calculation of P_e

Before post-tensioning, $P_e = 1.40 \times 10^5$ lb/in^2. The strain introduced at the level of pretension steel by post-tensioning is given by:

$$\varepsilon = \frac{\sigma}{E_c} = \frac{1}{E_c}\left[-\frac{P^1}{A} - \frac{P^1 e_2 C_1}{I}\right]$$

where E_c is the elastic modulus of concrete, and the remaining terms are as defined in section 3.B.1.6.

$$e_2 = -0.0211\,x^2 + 1.73\,x - 10.1, \quad \text{and} \quad x = 20.5$$

$$= -0.0211\,(20.5)^2 + 1.73\,(20.5) - 10.1$$

$$= 16.5"$$

and $$\varepsilon = \frac{1}{4.42 \times 10^6 \text{ psi}} \left[-\frac{6.02 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} - \frac{(6.02 \times 10^5 \text{ lb})\,(16.5 \text{ in})\,(28.15 \text{ in})}{5.92 \times 10^5 \text{ in}^4} \right]$$

$$= -1.81 \times 10^{-4}$$

so Δf pretension steel $= E_{st} \cdot \varepsilon = (29 \times 10^6 \text{ psi})\,(-1.81 \times 10^{-4})$

$$= -5.26 \times 10^3 \text{ psi}$$

The estimated pretension force after post-tensioning is therefore

$$P_e = [1.40 \times 10^5 \text{ psi} - 5.26 \times 10^3 \text{ psi}]\,(4.31 \text{ in}^2)$$

$$= 5.81 \times 10^5 \text{ lb}$$

3.B.1.8 Calculation of the Moments Due to the Loads

3.B.1.8.1 Dead-Load Moments

Guidance Rails and Supports

Quarter-span moment $= \frac{P\ell}{4} = \frac{(3{,}918 \text{ lb})\,(82.0')}{4} = 8.032 \times 10^4$ lb-ft

Levitation Strips

Quarter-span moment $= \frac{3\omega\ell^2}{32} = \left(\frac{3}{32}\right) \frac{(22.3 \text{ lb/ft})\,(82.0 \text{ ft})^2}{} = 1.41 \times 10^4$ lb-ft

Dead Weight of Guideway

Quarter-span moment $= \frac{3\omega\ell^2}{32} = \left(\frac{3}{32}\right)(1.91 \times 10^3 \text{ lb/ft})\,(82 \text{ ft})^2$

$$= 1.20 \times 10^6 \text{ lb-ft}$$

The total dead-load moment at quarter span, M_D, is therefore 1.29×10^6 lb-ft

3.B.1.8.2 Live-Load Moment

The live-load quarter-span moment, M_L, is therefore

$$\frac{3\omega\ell^2}{32} = \left(\frac{3}{32}\right)(1.214 \times 10^3 \text{ lb/ft})(82.0 \text{ ft})^2$$

$$= 7.65 \times 10^5 \text{ lb-ft}$$

The total quarter-span moment due to load is therefore

$$M_D + M_L = 1.29 \times 10^6 + 7.65 \times 10^5$$

$$= 2.06 \times 10^6 \text{ lb-ft}$$

$$= 2.47 \times 10^7 \text{ lb-in}$$

The top fibre stress at quarter-span is

$$f_1 = -\frac{P_e}{A} + \frac{P_e e_1 C_1}{I} - \frac{p^1}{A} + \frac{p^1 e_2 C_1}{I} - \frac{(M_D + M_L)\, C_1}{I}$$

$$= -\frac{5.81 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} + \frac{(5.81 \times 10^5 \text{ lb})(28.15 \text{ in})(21.1 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{6.02 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} + \frac{(6.02 \times 10^5 \text{ lb})(16.5 \text{ in})(21.1 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{(2.47 \times 10^7 \text{ lb-in})(21.1 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$= -589.8 \text{ psi}$$

The allowable compressive force is 2,700 psi, so the factor of safety is

$$\text{F.S.} = \frac{2{,}700}{589.8} = 4.6, \quad \text{which is satisfactory}$$

The bottom-fibre stress at quarter-span

$$f_2 = -\frac{P_e}{A} - \frac{P_e e_1 C_2}{I} - \frac{p^1}{A} - \frac{p^1 e_2 C_2}{I} + \frac{(M_D + M_L)\, C_2}{I}$$

$$= -\frac{5.81 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} - \frac{(5.81 \times 10^5 \text{ lb})(28.15 \text{ in})(30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$- \frac{6.02 \times 10^5 \text{ lb}}{1.83 \times 10^3 \text{ in}^2} - \frac{(6.02 \times 10^5 \text{ lb})(16.5 \text{ in})(30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$+ \frac{(2.47 \times 10^7 \text{ lb-in})\ (30.9 \text{ in})}{5.92 \times 10^5 \text{ in}^4}$$

$$= -729.3 \text{ psi}$$

Again, the allowable compressive force is 2,700 psi, and the safety factor is

$$\text{F.S.} = \frac{2,700}{729.3} = 3.7, \quad \text{again satisfactory}$$

The beam can easily sustain the comprised stresses.

3.B.1.9 Deflection Analysis

The deflection of the beam, y, is related to the bending moment by the following differential equation:

$$M(x) = \frac{E\ I\ d^2y}{dx^2}$$

where M(x) is the bending moment as a function of x. (Upward deflections are, by convention, considered positive).

M(x) can be disaggregated into the following components:

$$M(x)_{\text{total}} = M(x)_{\substack{\text{pretension} \\ \text{force}}} + M(x)_{\substack{\text{Post tension} \\ \text{force}}} + M(x)_{\text{Loads}}$$

$$= -P_e e_1 - P^1 e_2 + \frac{\left(\omega_{\text{dead}} + \omega^*_{\text{live}}\right)}{2} (\ell x - x^2)$$

Where

$P_e = 5.74 \times 10^5$ lb — This is the mid span value of P_e and is used to develop the mid span deflection only.

$P^1 = 6.02 \times 10^5$ lb

$e_1 = 28.15$ in

$e_2 = -1.47 \times 10^{-4} x^2 + .144x - 10.1$

$\omega_{\text{dead}} = 2,076$ lb/ft = 173 lb/in

$\omega^*_{\text{live}} = 101$ lb/in

$\ell = 82.0' = 984$ in

So that $$E_c I \frac{d^2y}{dx^2} = - (7.74 \times 10^5 \text{ lb}) (28.15 \text{ in})$$

$$- (6.02 \times 10^5 \text{ lb}) (-1.47 \times 10^{-4}x^2 + .144x - 10.1 \text{ in})$$

$$+ \left(\frac{274 \text{ lb/in}}{2}\right) (984x - x^2)$$

$$E_c I \frac{d^2y}{dx^2} = -1.62 \times 10^7 \text{ lb-in} + 88.5x^2 \text{ lb-in} - 8.67 \times 10^4 x \text{ lb-in}$$

$$+ 6.08 \times 10^6 \text{ lb-in} + 1.35 \times 10^5 x \text{ lb-in} - 137x^2 \text{ lb-in}$$

$$= -48.5x^2 \text{ lb-in} + 4.83 \times 10^4 x \text{ lb-in} - 1.01 \times 10^7 \text{ lb-in}$$

$E_c = 4.42 \times 10^6 \text{ lb/in}^2$ and $I = 5.92 \times 10^5 \text{ in}^4$, so

$$E_c I = \left(4.42 \times 10^6 \frac{\text{lb}}{\text{in}^2}\right) (5.92 \times 10^5 \text{ in}^4)$$

$$= 2.62 \times 10^{12} \text{ lb-in}^2$$

So that $$\frac{d^2y}{dx^2} = - \frac{48.5x^2 \text{ lb-in}}{2.62 \times 10^{12} \text{ lb-in}^2} + \frac{4.83 \times 10^4 x \text{ lb-in}}{2.62 \times 10^{12} \text{ lb-in}^2}$$

$$- \frac{1.01 \times 10^7 \text{ lb-in}}{2.62 \times 10^{12} \text{ lb-in}^2}$$

$$= -1.85 \times 10^{-11}x^2 \text{ in}^{-1} + 1.84 \times 10^{-8}x \text{ in}^{-1} - 3.85 \times 10^{-6} \text{ in}^{-1}$$

Let $A = -1.85 \times 10^{-11} \text{ in}^{-1}$

$B = 1.84 \times 10^{-8} \text{ in}^{-1}$

$C = -3.85 \times 10^{-6} \text{ in}^{-1}$

The equation can now be written as

$$\frac{d^2y}{dx^2} = Ax^2 + Bx + C$$

$$\frac{dy}{dx} = \frac{Ax^3}{3} + \frac{Bx^2}{2} + Cx + D_1$$

and $$y = \frac{Ax^4}{12} + \frac{Bx^3}{6} + \frac{Cx^2}{2} + D_1 x + D_2$$

where D_1 and D_2 are integration constants.

Examining the boundary conditions:

$$y_{/x = 0} = 0; \quad \text{so that } D_2 = 0$$

and $$y_{/x = \ell = 984} = 0; \quad \text{so that } D_1 = -\frac{A\ell^3}{12} - \frac{B\ell^2}{6} - \frac{C\ell}{2}$$

$$y = \frac{A}{12}x^4 + \frac{B}{6}x^3 + \frac{C}{2}x^2 - \left(\frac{A\ell^3}{12} + \frac{B\ell^2}{6} + \frac{C\ell}{2}\right)x$$

$$y_{/x = \frac{\ell}{2}} = \frac{A\ell^4}{192} + \frac{B\ell^3}{48} + \frac{C\ell^2}{8} - \frac{A\ell^4}{24} \quad \frac{B\ell^3}{12} \quad \frac{C\ell^2}{4}$$

$$= A\ell^4\left(\frac{-7}{192}\right) + B\ell^3\left(-\frac{1}{16}\right) + C\ell^2\left(-\frac{1}{8}\right)$$

$$= \frac{-7\ (-1.85 \times 10^{-11} \text{in}^{-1})\ (984 \text{ in})^4}{192 \text{ in}^2} - \frac{(1)\ (1.84 \times 10^{-8} \text{ in}^{-1})\ (984 \text{ in})^3}{16 \text{ in}}$$

$$- \frac{(1)\ (-3.85 \times 10^{-6} \text{ in}^{-1})\ (984 \text{ in})^2}{8}$$

which reduces to

$$y_{/x = \frac{\ell}{2}} = +.632 \text{ in} - 1.096 \text{ in} + .466 \text{ in}$$
$$= .002 \text{ in} = .005 \text{ cm} \qquad \text{which is acceptable.}$$

The beam is adequate with respect to strength and servicability in bending (shear requirements have not been considered).

3.B.1.10 GUIDEWAY COMPATABILITY WITH REVISED LOADINGS

The redistribution of magnets will reduce the applied bending moments--implying a reduction in section size. Consideration must, however, be given to the mode of operation that utilizes rubber wheel contact only. Conceivably this mode of operation could occur over any section of guideway, due to emergency procedures or total malfunction, etc. However, in the wheeled mode, with the vehicle live loading applied to the guideway as four point loads, applied in pairs, 22.2 m apart, and applying an impact factor of 25%, the live load moment M_L does not exceed the value used as the basic design live load moment.

APPENDIX B

DESIGN OF BOX SECTION GIRDER

Consider a minimum depth box section girder as shown in Figure 4.2 with a span of 25 m.

Section Properties

Area (A) = $(2.6 + 2 \times 1.2 + 2.0)\ (0.15) = 1.05\ m^2$

Height of centroid above soffit:

$$y_b = (2.6 \times 0.15 \times 1.275 + 2 \times 1.2 \times 0.15 \times 0.6 + 2.0 \times 0.15 \times 0.075)/A$$

$$= 0.70\ m$$

Second moment of area about horizontal centroid axis:

$$I_x = \frac{2.6 \times 0.15^3}{12} + 2.6 \times 0.15 \times (0.575)^2 + 2 \times \frac{0.15 \times (1.2)^3}{12} + 2 \times 0.15 \times 1.2 \times (0.1)^2 + \frac{2.0 \times (0.15)^3}{12} + 2.0 \times 0.15 \times (0.625)^2$$

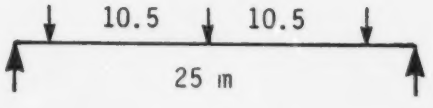
$$= 0.294\ m^4$$

Second moment of area about vertical centroid axis:

$$I_y = \frac{0.15 \times (2.6)^3}{12} + \frac{0.15 \times (2.0)^3}{12} + 2\left(\frac{1.2 \times (0.15)^3}{12} + 1.2 \times 0.15 \times (1.025)^2\right)$$

$$= 0.736\ m^4$$

Loading



Dead load = 1.05 x 2.4 x 9.8 = 24.7 kN/m

+ 1.2 kN/m for stator

= 25.9 kN/m

= 26 kN/m (say)

Live load = 10.95 (Table 1) x 10.5 = 115.0 kN x 1.8 (impact)

= 207 kN

Mid-span moments:

$$M_D = \frac{w\ell^2}{8} = \frac{26 \times 25^2}{8} = 2031 \text{ kN.m}$$

$$M_L = \frac{W\ell}{4} = \frac{207 \times 25}{4} = 1294 \text{ kN.m}$$

Try a prestressing force, P = 3300 kN, with an eccentricity

e = 700 - 60 = 640 mm

Stress in top fibre:

$$\sigma_T = -\frac{3300 \times 10^3}{1.05 \times 10^6} + \frac{3300 \times 10^3 \times 640 \times 640}{0.294 \times 10^{12}} - \frac{2031 \times 10^6 \times 650}{0.294 \times 10^{12}} - \frac{1294 \times 10^6 \times 650}{0.294 \times 10^{12}} \text{ (MPa)}$$

$$= -3.14 + 4.67 - 4.49 - 2.86$$

$$= -5.82 \text{ MPa}$$

Stress in bottom fibre:

$$\sigma_T = 3.14 - 4.67 \times \frac{700}{650} + 4.49 \times \frac{700}{650} + 2.86 \times \frac{700}{650}$$

$$= -3.14 - 5.03 + 4.83 + 3.08$$

$$= -0.26 \text{ MPa} \qquad \text{small compression} \qquad \text{O.K.}$$

Assume final stress level of 60 percent ultimate in prestressing steel:

$$P_{ult} = \frac{3300}{0.6} = 5500 \text{ kN}$$

Using Size 15 1860 strand requires:

$$\frac{5500}{261} = 21 \text{ strands}$$

Use 22 strands with 60 mm spacing.

Check end stress:

Assume 75 percent ultimate at transfer.

$$P = 0.75 \times 261 \times 22 = 4306 \text{ kN}$$

$$\sigma_T = -\frac{4306 \times 10^3}{1.05 \times 10^6} + \frac{4306 \times 10^3 \times 640 \times 650}{0.294 \times 10^{12}}$$

$$= -4.10 + 6.09 = +1.99 \text{ Mpa} > 1.31 \text{ (see below)} \qquad \text{N.G.}$$

Note for 40 MPa concrete with 30 MPa at transfer:

$$f_{cri} = 0.6\sqrt{30} = 3.28 \text{ MPa}$$

$$0.4\, f_{cri} = 0.4 \times 3.28 = 1.31 \text{ MPa}$$

Add two size 13 strands to top slab:

$$P = 0.6 \times 261 \times 2 = 313 \text{ kN}$$

$$\sigma_T = -\frac{313 \times 10^3}{1.05 \times 10^6} - \frac{313 \times 10^3 \times 590 \times 650}{0.294 \times 10^{12}}$$

$$= -0.30 - 0.41 = -0.71 \text{ MPa}$$

Final stress in top fibre:

$\sigma_T = + 1.99 - 0.71 = 1.28 \quad 1.31$ O.K.

Check mid-span stress at transfer:

Assume stress level of 72 percent ultimate in prestressing steel.

$P_B = 0.72 \times 261 \times 22 = 4134$ kN

$P_T = 0.72 \times 184 \times 2 = 265$ kN

4399 kN

$$\sigma_T = - \frac{4399 \times 10^3}{105 \times 10^6} + \frac{4134 \times 10^3 \times 640 \times 650}{0.294 \times 10^{12}} - \frac{265 \times 10^3 \times 590 \times 650}{0.294 \times 10^{12}} - \frac{2031 \times 10^6 \times 650}{0.294 \times 10^{12}}$$

$= - 4.19 + 5.85 - 0.85 - 4.49$

$= - 3.18$ MPa compressive O.K.

Check final stress at mid-span:

Assume stress level of 60 percent ultimate in prestressing steel.

$P_B = 0.60 \times 261 \times 22 = 3445$ kN

$P_T = 0.60 \times 184 \times 2 = 221$ kN

3,666 kN

$$\sigma_T = - \frac{3666 \times 10^3}{1.05 \times 10^6} + \frac{3445 \times 10^3 \times 640 \times 650}{0.294 \times 10^{12}} - \frac{221 \times 10^3 \times 590 \times 650}{0.294 \times 10^{12}} - \frac{2031 \times 10^6 \times 650}{0.294 \times 10^{12}} - \frac{1294 \times 10^6 \times 650}{0.294 \times 10^{12}}$$

$= - 3.49 + 4.87 - 0.29 - 4.48 - 2.86$

$= - 6.26$ MPa $< 0.45 \times 40 = 18$ O.K.

$$\sigma_B = -2.49 - 4.87 \times \frac{700}{650} + 0.25 \times \frac{700}{650} + 4.49 \times \frac{700}{650}$$

$$+ 2.86 \times \frac{700}{650}$$

$$= - 3.49 - 5.24 + 0.31 + 4.83 + 3.08$$

$$= - 0.51 \text{ MPa} \qquad \text{O.K.}$$

Effect of centrifugal force:

Centrifugal force (CF) = 20 percent vertical live load

Mid-span moment due to CF = 0.2 x 1294

= 259 kN.m

Stress due to CF:

$$\sigma_T = \pm \frac{259 \times 10^6 \times 1300}{0.736 \times 10^{12}} = \pm 0.46 \text{ MPa}$$

$$\sigma_B = \pm \frac{259 \times 10^6 \times 1150}{0.736 \times 10^{12}} = \pm 0.40 \text{ MPa}$$

Superimpose these stresses on stresses at mid-span:

$\sigma_T = - 6.26 - 0.46 = - 6.72 \text{ MPa} < 18$ O.K.

$\sigma_B = - 0.51 + 0.40 = - 0.11 \text{ MPa}$

Effect of longitudinal braking force:

Longitudinal force (LF) = 30 percent vertical live load

LF on single girder = 0.3 x 10.95 x 25 = 82 kN

Moment due to bearing resistance = 82 x 0.7 = 57.5 kN.m

Resultant stress at end of beam:

$$\sigma_T = - \frac{82 \times 10^3}{1.05 \times 10^6} + \frac{57.5 \times 10^6 \times 650}{0.294 \times 10^{12}}$$

$$= - 0.08 + 0.13 = + 0.05 \text{ MPa}$$

$$\sigma_B = - 0.08 - 0.13 \times \frac{700}{650} = - 0.22 \text{ MPa}$$

These stresses are negligible.

Thus service load combination

$$SI = D + L + I + PS + LF + CF$$

is satisfied.

Service load combination S2 (= SI + 0.3 WL):

Wind load (WL) = 5.29 kN/m

$$\text{Moment} = \frac{5.29 \times 25^2}{8} = 413 \text{ kN.m}$$

$$T = \pm \frac{413 \times 10^6 \times 1300}{0.736 \times 10^2} = \pm 0.73 \text{ MPa}$$

$$B = = \frac{413 \times 10^6 \times 1150}{0.736 \times 10^{12}} = \pm 0.65 \text{ MPa}$$

Superimpose on load combination SI stresses at mid-span:

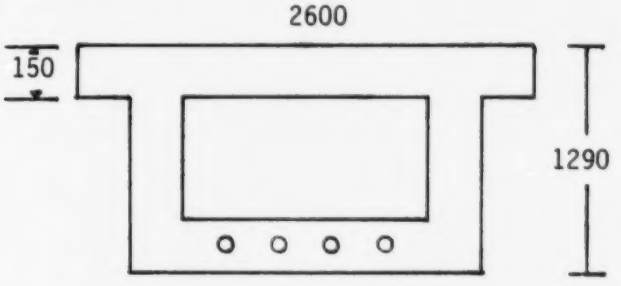
$T = - 6.72 - 0.73 = - 7.45 \text{ MPa} < 18$ O.K.

$B = - 0.11 + 0.65 = + 0.54 \text{ MPa}$

Thus, service load combination S2 is satisfied.

Service load combinations S3 and S4 will not be critical in this case.

Compute Flexural Strength of Section



Ignore two stands in top flange.

$A_{ps} = 22 \times 140 = 3080 \text{ mm}^2$

$f_{pu} = 1860$ MPa (low relaxation)

$f_c = 40$ MPa

Assume neutral axis in flange.

$$\rho_p = \frac{A_{ps}}{bd} = \frac{3080}{2600 \times 1290} = 0.0009$$

$$w_p = \rho_p f_{pu}/f'_c = 0.0009 \times \frac{1860}{40} = 0.043 < 0.3 \ (\text{under-reinforced})$$

$$f_{ps} = 1860 (1 - 0.5 \times 0.043) = 1820 \text{ MPa}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{3080 \times 1820}{0.85 \times 40 \times 260} = 63.4 \text{ mm} < 150 \ (\text{NA in flange})$$

$$M_n = A_{ps} f_{ps} (d - a/2)$$

$$= 3080 \times 1820 (1290 - 63.4/2) \ 10^{-6}$$

$$= 7053 \text{ kN.m}$$

$$\phi M_n = 0.9 \times 7053 = 6348 \text{ kN.m}$$

Required capacity:

$$U4 = 1.3 D + 1.4 L$$

$$= 1.3 \times 2031 + 1.4 \times 1294$$

$$= 4452 \text{ kN.m} < 6348 \quad \text{O.K.}$$

Compute Cracking Moment

$$f_r = \frac{P}{A} + \frac{Pey}{I} + \frac{M_{cr} y}{I}$$

$$+ 0.6 \sqrt{40} = - 3.49 - 5.24 + 0.31 + \frac{M_{cr} \times 700}{0.294 \times 10^{12}}$$

$$M_{cr} = 5100 \text{ kN.m}$$

$$1.2 M_{cr} = 6200 \text{ kN.m} < 6358 \quad \text{O.K.}$$

Estimate Camber

Centroid of prestress:

$$= \frac{198 \times 60 + 3080 \times 1290}{3278}$$

$$= 1216 \text{ from top surface}$$

Eccentricity = 1216 - 650 = 566

P at transfer = $0.72 \times 1860 \times (198 + 3080)\ 10^{-3}$ = 4390 kN

Upward deflection due to prestress:

$$= \frac{M\ell^2}{8EI}$$

$$= \frac{4390 \times 10^3 \times 566 \times 25^2 \times 10^6}{8 \times 27{,}700 \times 0.294 \times 10^{12}}$$

= 24 mm.

Downward deflection due to mass:

$$= \frac{5\ w\ell^4}{384\ EI}$$

$$= \frac{5 \times 24.7 \times 25^4 \times 10^{12}}{384 \times 27{,}700 \times 0.294 \times 10^{12}}$$

= 15 mm

Net camber at release = 24 - 15 = 9 mm

At erection: camber = 1.85* x 24 - 1.8* x 15 = 17 mm

Final: camber = 2.70* x 24 - 2.45* x 15 = 28 mm

Final camber of 28 mm exceeds maximum value of 19 mm specified in Table 4.4. However, final camber will be reduced by any superimposed dead load applied to guideway and it appears that the Maglev vehicle can tolerate such a camber. It is suggested that camber control be investigated in the final design stage.

Design for Shear

Check capacity of minimum shear reinforcement:

$$A_v = \frac{A_{ps}}{80} \cdot \frac{f_{pu}}{f_y} \cdot \frac{S}{d} \sqrt{\frac{d}{b}}$$

$$= \frac{3080}{80} \cdot \frac{1860}{400} \cdot \frac{S}{1290} \cdot \sqrt{\frac{1290}{300}}$$

$$= 0.288\ S$$

Spacing $S \leq 0.75\ h = 0.75 \times 1350 = 1012$ mm

$3\ b_w = 3 \times 150 = 450$ mm governs

600 mm $= 600$ mm

Therefore $A_v = 0.288 \times 450 = 130$ mm^2

Using 10 M stirrups with one leg in each web gives 200 mm^2

Therefore, $V_s = \frac{\phi_s\ A_v\ f_{yd}}{S} = \frac{0.85 \times 200 \times 400 \times 1290}{450}$ (N)

Compute concrete contribution:

$$V_c = (0.06 \sqrt{f'_c} + 6 \frac{V_f}{M_f} d_p)\ \theta_c\ b_w\ d$$

$$= (0.06 \sqrt{40} + 6 \frac{V_f}{M_f} 1290)\ 0.6 \times 300 \times 1290\ (N)$$

$$= (0.38 + 7.74 \frac{V_f}{M_f})\ 282.2\ kN$$

---

* Metric Design Manual, Canadian Prestressed Concrete Institute, First Edition, 1982 (Fig. 3.5.3).

Values of V_c at different locations along the span are given in Table A.1.

TABLE A.1

SHEAR CAPACITIES OF SECTION

x (m)	V_f/M_f (1/m)	$V_f\ dp/M_c$	V_c (kN)	V_s (kN)	$V_r = (V_c+V_s)$ (kN)	V_f (kN)
0	-	1.000	1,065*	195	1260	767
2	.456	0.588	908	195	1103	644
4	.202	0.261	451	195	646	521
5	.114	0.147	294†	195	489	398
6	.066	0.077	294†	195	489	275
10	.033	0.043	294†	195	489	152
12	.006	0.008	294†	195	489	29

* Max $V_c = 0.4 \sqrt{f'_c}\, b_w\, d = 0.4 \sqrt{40} \times 300 \times 1290 = 979$ kN

† Min $V_c = 0.17 \sqrt{f'_c}\, b_w\, d = 0.17 \sqrt{40} \times 300 \times 1290 = 416$ kN

The variations of V_r and V_f along the span are shown in Figure A.1.

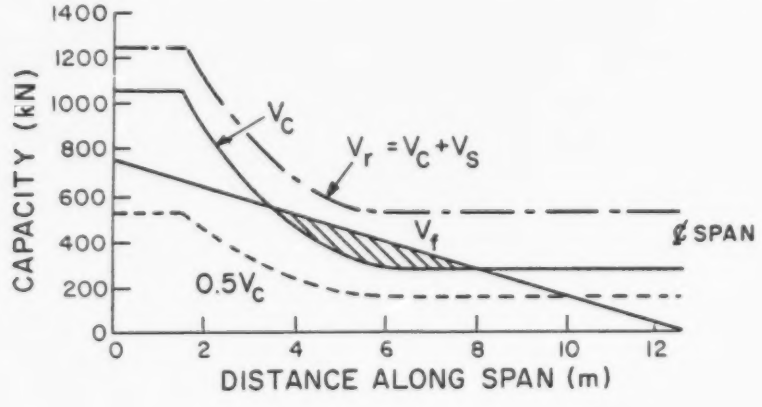


Figure A.1 Shear variation along span

Hence, 10 m U stirrups at 450 mm are adequate shear reinforcement over the length of the span.

Torsion Design

Torsional moment due to:

centrifugal force (CF) = 1.96 kN.m/m

wind loading (WL) = 8.46 kN.m/m

destabilization (DS) = 1.81 kN.m/m

Strength load combination:

$U1 = 1.3\ D + 1.4\ (L + CF) + 1.5\ (WL + DS)$

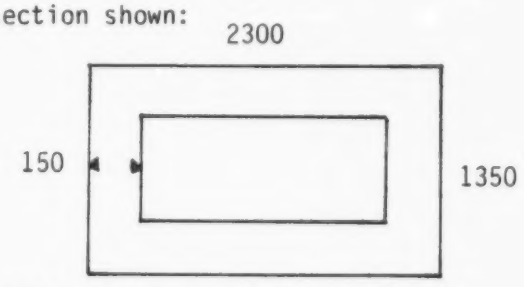
$= 1.4 \times 1.96 + 1.5\ (8.46 + 1.81)$

$= 18.15$ kN.m/m

Compute cracking torque, T_{cr}:

$$T_{cr} = (A^2_c/p_c)\ 0.4\ \phi_c\ \sqrt{f'_c}\ \sqrt{1 + f_{pc}/0.4\ \phi_d\ \sqrt{f'_c}}$$

Assume box section shown:



$A_c = 2300 \times 1350 = 3.105 \times 10^6$ mm^2

$p_c = (2300 + 1350)\ 2 = 7300$ mm

$$T_{cr} = \frac{3.105^2 \times 10^{12}}{7300} \times 0.4 \times 0.6 \sqrt{40}$$

$$\frac{}{1 + \sqrt{\frac{3.47}{0.4 \times 0.6 \sqrt{40}}}} \quad \text{(N.min)}$$

$= 3600$ kN.m

$0.25\ T_{cr} = 900$ kN.m

$T_f = 18.15 \times 25/2 = 227 \text{ kN.m} < 0.25\ T_{cr}$

Therefore, torsion can be neglected.

Note that torsional capacity of box section is much greater than required capacity. Consequently, an open section rather than a closed box may be suitable.

Reinforcement in Top Slab

Loads from skid pads in emergency braking condition will be carried into web of the box section by arch section and thus minimum flexural reinforcement should suffice in top slab.

Minimum slab reinforcement = 0.002 Ag

= 0.002 x 1000 x 150

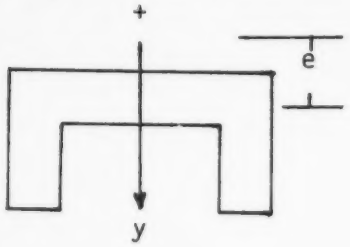
= 300 mm^3/m

Use 10 m at 300 mm.

APPENDIX C

DESIGN OF CHANNEL-SECTION GIRDER

Consider the channel section girder as shown in Figure 4.3 with a span of 30 m.



Location of shear centre:

Shear centre is located* at a distance, e, given by:

$$e = \frac{a^2 b^2 t}{I_y}$$

$$I_y = \frac{220 \times 2600^3}{12} + 2\left[\frac{1630 \times 220^3}{12} + 1850 \times 220 \times 1190^2\right]$$

$$= 1{,}4668 \times 10^{12}\ mm^7$$

$$a = 1300 - 110 = 1190 \qquad b = 1850 - 110 = 1740$$

$$e = \frac{1190^2 \times 1740^2 \times 220}{1.4778 \times 10^{12}} = 638\ mm$$

Shear centre is located at 638 - 110 = 528 mm above top surface of guideway.

Torstional loading:

Compute torque by using eccentricity of side load with respect to shear entre.

Centrifual force torque

= 1.09 x (1.8 - 0.6 - 0.528) = 0.732 kN.m/m

Wind loading torque

= 5.29 x (1.6 - 0.6 - 0.528) = 2.50 kN.m/m

Destabilization force torque

= 1.81 x (1.0 - 0.6 - 0.528) = 00.232 kN.m/m

---

* Budynas, R.G. Advanced Strength and Applied Stress Analysis. McGraw-Hill, 1977, p. 116.

Load combinations:

$$UI = 1.4\ CF + 1.5\ (WL + DS)$$

$$= 1.4 \times 0.732 + 1.5\ (2.50 + 0.232)$$

$$= 5.12\ kN.m/m$$

Therefore, $T_f = 5.12 \times 30/2 = 76.8\ kN.m$

Cracking torque:

$$A_c = 220 \times 1850 \times 2 + 2160 \times 220 = 1{,}289{,}200\ mm^2$$

$$b_c = 1850 \times 2 + 2600 + 1630 \times 2 + 2160 = 11{,}720\ mm$$

$$f_{pc} = 3\ MPa \qquad (say)$$

$$T_{cr} = \frac{1289\ 200^2}{11720} \times 0.4 \times 0.6\sqrt{40}\sqrt{1 + 3/0.4 \times 0.6\sqrt{40}}\ (N.mm)$$

$$= 371\ kN.m$$

$$0.25\ T_{cr} = 92.7\ kN.m > 76.8$$

Torsion may be neglected according to cluase 11.2.4.1 of A23.3-M84. Consequently, torsion does not appear to be a problem with this open secion.

Design of the channel section for flexure and shear is similar to that of the box section and is not repeated.

APPENDIX D

SUBMISSION BY SCHOKBETON QUEBEC INC.

SCHOKBETON QUEBEC INC.

P.O. Box 240 St. Eustache P.Q. (514) 473-6831
J7R 4K8

April 25, 1986

Canadian Institute of Guided
Ground Transport
Queen's University
Kinston, Ontario
K7L 3N6

Attention: Mr. T.I. Campbell, P. Eng.
Research Fellow, CIGGT

Re: Costing Study of the Revised
Canadian Maglev Guideway

Dear Sir:

Following your letter dated April 1st, 1986 and our meeting held at Schobeton's office on April 9th in the presence of Mr. Christopher J. Boon, we are pleased to supply you attached herewith, our cost estimates for the Box section as per figure 2 and for the channel section as per figure 3.

For comparative purposes, we have assumed both sections being 25 metres long. As discussed during our meeting of April 9th, we did not estimate the original section as you already had cost estimates made for that section.

The base estimates are based on a 10 year production program using current material costs and plant labour rates: as we do not wish to disclose our overhead and profit margin, we have used a 42% mark up which represents the average mark up of the Precast Concrete industry in America. This mark up includes profit, office and plant overhead as well as all plant equipment, supervision and inspection personnel.

The base estimates do not include the provincial sales tax which differ in Quebec from Ontario.

In Quebec, a 9% sales tax apply on material cost only if the product is erected by the supplier or on the selling price, if not erected by the supplier. In Ontario, the 7% sales tax is applied on the F.O.B. selling price despite the fact that the product is or not erected by the supplier.

. . ./2

On a separate sheet, we have outlinede the additional costs applicable to each type of unit, should the production periode be reduced to a 6 year production program and should the reinforcing steel be changed to epoxy coated steel.

You will also find attached herewith our invoice in the amount of $4,000.00 covering our cost for the preparation of these estimates.

Should your require additional information, please feel free to call on the undersigned.

Yours truly,

SCHOKBETON QUEBEC INC.

Lino Pansieri
Directeur des Ventes

LP/mg
Encl.

ANNEX

REVISED CANADIAN MAGLEV GUIDEWAY

PRICE ADJUSTMENT FOR EPOXY COATED STEEL

Epoxy coated steel	$1.43/kg
Standard reinforcing steel	.51/kg
Difference	0.92/kg
mark up 42%	.39/kg
Total	1.31/kg
8% Federal Sales Tax	.11/kg
	$1.42/kg extra

Box section (fig. 2)

981 kg x $1.42 = $1,393.00 extra/unit
= 55.72 extra/lin. metre
= 41.09 extra/cu. metre

Channel section (fig. 3)

1,304 kg x $1.42 = $1,852.00 extra/unit
= 74.08 extra/lin. metre
= 48.96 extra/cu. metre

PRICE ADJUSTMENT FOR 6 YEAR PRODUCTION PROGRAMM

Box section (fig. 2)

Additional form costs	$14.85 + $163.35 =	$178.20/unit
42% mark up	=	74.85
total		$253.05
8% Federal Sales tax		20.25
Total extra per unit	=	273.30
per lin. metre	=	10.94
per cu. metre	=	8.06

Channel section (fig. 3)

Additional form costs	$15.45 + $169.95 =	$185.40
42% mark up	=	77.87
total		$263.27
8% Federal Sales tax	=	21.06
Total extra per unit	=	284.33
per lin. metre	=	11.38
per cu. metre	=	7.52

SCHOKBETON QUEBEC INC.

CA-MAGLEV-GUIDEWAY
0

PRICE-ANALYSIS

**BOXBEAM(fig2)

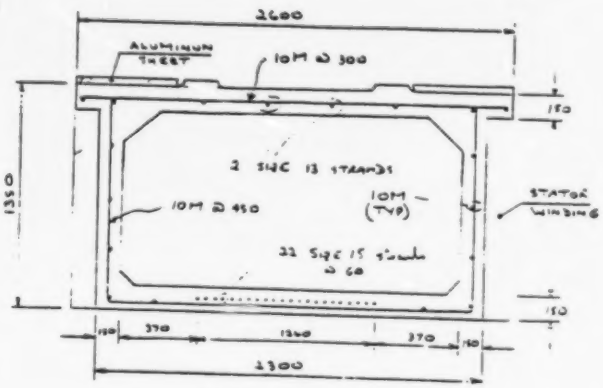
TEN/YR-PROD-PERIOD

11750-BEAMS/PERIOD

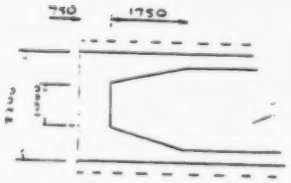
.....8-MOLDS-REQ,D
0.8
0

16PCS/3DAYS-PROD

1175PCS/YR-PROD



CROSS SECTION



END SECTION

SPAN = 25 m
f'_c = 40 MPa
f_{pu} = 1860 MPa
f_y = 400 MPa
47,000 BEAMS

FIG. 2
CANADIAN MAGLEV GUIDEWAY BOX BEAM

**BOXBEAM(fig2)

REGISTERED LIST
1 *LENGTH(m)
25.00
2 DEV-AREA(sq m
205.00
3 VOLUME(cu m)
33.90
4 WEIGHT(m ton)
81.50

**BOXBEAM(fig2)

PRICE-Fab-PLANT-86
0

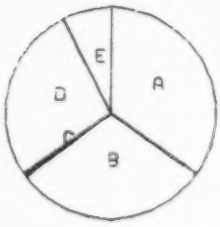
HOURLY-RATE:$15.25
0

($)/UNIT

REGISTERED LIST
1 *LABOUR
5,321.00
2 *MATERIALS
4,605.00
3 *ENG
100.00
4 O.H&PROF-42%
4,211.00
5 *8% FED TAX
1,139.00

TOTAL:

15,376.00



A LABOUR
..... 34.61%
B MATERIAL
..... 29.95%
C ENG
..... 0.65%
D O.H&PROF-4
..... 27.39%
E 8% FED TAX
..... 7.40%

$15,376.00/UNIT

$615.04/LINEAR METER

$453.57/CUBIC METER

PROV-TAX-EXTRA

SCHOKBETON QUEBEC INC.

**BOXBEAM(fig2)

LABOUR($/UNIT)

REGISTERED LIST

1	*10(mold)sq m	297.00
2	*15sp form-hr	15.00
3	*30(R/B)Kg	981.00
4	*35(HARD)Kg	82.00
5	*40(setup)cu m	33.90
6	*50(conc)cu m	33.90
7	*60(finish)hr	13.50
8	*70(storag)hr	7.00
9	*80(loading-hr	7.00

**BOXBEAM(fig2)

MATERIAL($/UNIT)

REGISTERED LIST

1	*MOLD(sq m)	297.00
2	*15FORM(sq m)	50.00
3	*R/B (Kg)	981.00
4	*15mm STRAN(m	600.00
5	*L HOOKS(Qty)	4.00
6	galv HARD(Kg)	46.00
7	CONCRET(cu m)	34.90
8	U/MAT(cu m)	34.90

* DETAILS *

*10(mold)sq m

@	297.00	
*	0.05	
=	14.85	
		14.85

*15sp form-hr

@	15.00	
*	15.25	
=	228.75	
		228.75

*30(R/B)Kg

@	981.00	
*	0.69	
=	676.89	
		676.89

*35(HARD)Kg

@	82.00	
*	1.12	
=	91.84	
		91.84

*40(setup)cu m

@	33.90	
*	98.97	
=	3,355.08	
		3,355.08

*50(conc)cu m

@	33.90	
*	15.75	
=	533.92	
		533.92

*60(finish)hr

@	13.50	
*	15.25	
=	205.87	
		205.87

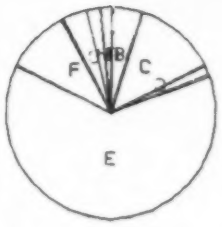
*70(storag)hr

@	7.00	
*	15.25	
=	106.75	
		106.75

*80(loading-hr

@	7.00	
*	15.25	
=	106.75	
		106.75

TOTAL 5,320.71



A	10(mold)	0.28%
B	15 sp form	4.30%
C	30(R/B)	12.72%
D	35(HARD)	1.73%
E	40(SET UP)	63.06%
F	50(CONCRET	10.03%
G	60(FINISH)	3.87%
H	70(STORAGE	2.01%
I	80(LOADING	2.00%

* DETAILS *

*MOLD(sq m)

@	297.00	
*	0.55	
=	163.35	
		163.35

*15FORM(sq m)

@	50.00	
*	6.68	
=	334.00	
		334.00

*R/B (Kg)

@	981.00	
*	0.51	
=	500.31	
		500.31

*15mm STRAN(m

@	600.00	
*	0.79	
=	474.00	
		474.00

*L HOOKS(Qty)

@	4.00	
*	10.00	
=	40.00	
		40.00

galv HARD(Kg)

@	46.00	
*	1.44	
=	66.24	
		66.24

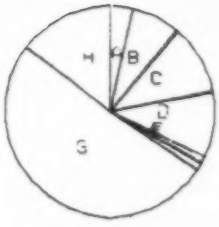
CONCRET(cu m)

@	34.90	
*	67.10	
=	2,341.79	
		2,341.79

U/MAT(cu m)

@	34.90	
*	19.62	
=	684.73	
		684.73

TOTAL 4,604.42



A	MOLD	3.55%
B	SP. FORM	7.25%
C	R/B	10.87%
D	15mm STRAN	10.29%
E	L. HOOKS	0.87%
F	galv HARD	1.44%
G	CONCRETE	50.86%
H	U/MAT	14.87%

SCHOKBETON QUEBEC INC.

CA-MAGLEV-GUIDEWAY
0

PRICE-ANALYSIS

CHANNEL-SECT(fig3)

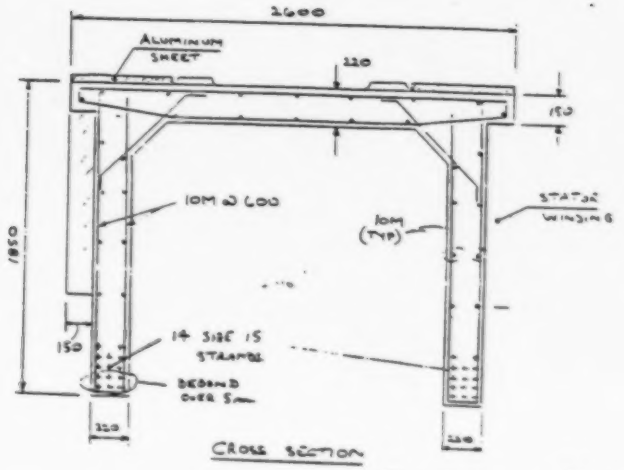
TEN/YR-PROD-PERIOD

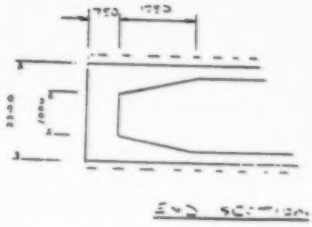
11750-BEAMS/PERIOD

8-MOLDS-REQ, D
9
0

:6PCS/3DAYS-PROD

1175PCS/YR-PROD





SPAN = 25m
f'_c = 42 MPa
f_{pu} = 1860 MPa
f_y = 400 MPa
39,000 BEAMS

FIG 3
CANADIAN MAGLEV GUIDEWAY
CHANNEL BEAM

CHANNEL-BEAM(fig3)

REGISTERED LIST
1 *LENGTH(m)
25.00
2 DEV-AREA(sq m
166.00
3 VOLUME(cu m)
37.83
4 WEIGHT(m ton)
98.88

CHANNEL-BEAM(fig3)

PRICE-FAB-PLANT-RE
0

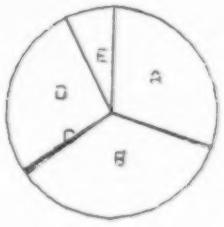
HOURLY-RATE:$15.25
0

($)/UNIT

REGISTERED LIST
1 *LABOUR
4,255.00
2 *MATERIALS
4,876.00
3 *ENG
100.00
4 O.H&PROF-42%
3,877.00
5 *8% FED TAX
1,049.00

TOTAL:

14,157.20



A LABOUR
..... 30.06%
B MATERIAL
..... 34.44%
C ENG
..... 0.71%
D O.H&PROF-4
..... 27.38%
E 8% FED TAX
..... 7.40%

$14,157.20 IN.

$566.28/LIN-METER

$374.23/CU-METER

PROV-TAX-EXTRA

SCHOKBETON QUEBEC INC.

CHANNEL-BEAM(fig3)

LABOUR($/UNIT)

REGISTERED LIST

1	*10(mold)sq m	309.00
2	*15sp form-hr	0.00
3	*30(R/B)Kg	1,304.00
4	*35(HARD)Kg	82.00
5	*40(setup)cu m	37.83
6	*50(conc)cu m	37.83
7	*60(finish)hr	13.50
8	*70(storag)hr	7.00
9	*80loading-hr	7.00

* DETAILS *

*10(mold)sq m
@ 309.00
* 0.05
= 15.45
15.45

*30(R/B)Kg
@ 1,304.00
* 0.52
= 678.08
678.08

*35(HARD)Kg
@ 82.00
* 1.12
= 91.84
91.84

*40(setup)cu m
@ 37.83
* 66.52
= 2,516.45
2,516.45

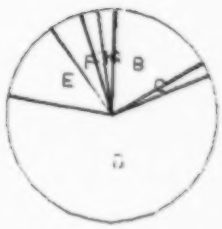
*50(conc)cu m
@ 37.83
* 14.11
= 533.78
533.78

*60(finish)hr
@ 13.50
* 15.25
= 205.87
205.87

*70(storag)hr
@ 7.00
* 15.25
= 106.75
106.75

*80loading-hr
@ 7.00
* 15.25
= 106.75
106.75

TOTAL
4,254.97



A 10(mold) 0.36%
B 30(R/B) 15.94%
C 35(HARD) 2.16%
D 40(SET UP) 59.14%
E 50(CONCR) 12.54%
F 60(FINISH) 4.84%
G 70(STORA) 2.51%
H 80(LOADING 2.51%

CHANNEL-BEAM(fig3)

MATERIAL($/UNIT)

REGISTERED LIST

1	*MOLD(sq m)	309.00
2	*R/B (Kg)	1,304.00
3	*15mm STRAN(m	700.00
4	*L HOOKS(Qty)	4.00
5	galv HARD(Kg)	46.00
6	CONCRET(cu m)	39.00
7	U/MAT(cu m)	39.00

* DETAILS *

*MOLD(sq m)
@ 309.00
* 0.55
= 169.95
169.95

*R/B (Kg)
@ 1,304.00
* 0.51
= 665.04
665.04

*15mm STRAN(m
@ 700.00
* 0.79
= 553.00
553.00

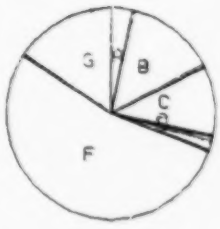
*L HOOKS(Qty)
@ 4.00
* 10.00
= 40.00
40.00

galv HARD(Kg)
@ 46.00
* 1.44
= 66.24
66.24

CONCRET(cu m)
@ 39.00
* 67.10
= 2,616.90
2,616.90

U/MAT(cu m)
@ 39.00
* 19.62
= 765.18
765.18

TOTAL
4,876.31



A MOLD 3.49%
B R/B 13.64%
C 15mm STRAN 11.34%
D L. HOOKS 0.82%
E galv HARD 1.36%
F CONCRETE 53.67%
G U/MAT 15.68%

SCHOKBETON QUEBEC INC.

SCHOKBETON QUEBEC INC.

P.O. Box 240 St. Eustache P.Q. (514) 473-6831
J7R 4K8

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QUEEN'S UNIVERSITY

May 28th, 1986

Canadian Institute of Guided
Ground Transport
Queen's University
KINSTON (Ontario)
K7L 3N6

Attention: Mr. T. I. Campbell, P. Eng.
Research Fellow, CIGGT

Re: Costing Study of the Revised
Canadian Maglev Guideway

Gentlemen:

As requested, please find attached herewith our cost estimate for the original section based on a 10 year production program.

For a 6 year production program, the cost per unit should be increased as follows:

$16.24 + $64.96	=	$81.20
42% Mark Up	=	34.10
TOTAL		$115.30
8% F.S.T.		9.22
TOTAL EXTRA COST per unit		$124.52
per lin. metre		4.98
per cu. metre		5.57

For epoxy coated steel, the applicable extra cost per unit is as follows:

1610 kg x $1.42 = $2,286.20 per unit.

Please note that we have substantially increased the rebar content by using 15 m + 10 m bars in lieu of #2 shown on the sketch.

. . ./2

These pieces do not include the deck slab nor the post tensioning material requested.

Hoping the present to your entire satisfaction, we remain,

Yours truly,

SCHOKBETON QUEBEC INC.

Lino Pansieri
Sales Director

LP/mg
Encl.

CA-MAGLEV-GUIDEWAY
0

PRICE-ANALYSIS

**U-SECT(fig1)

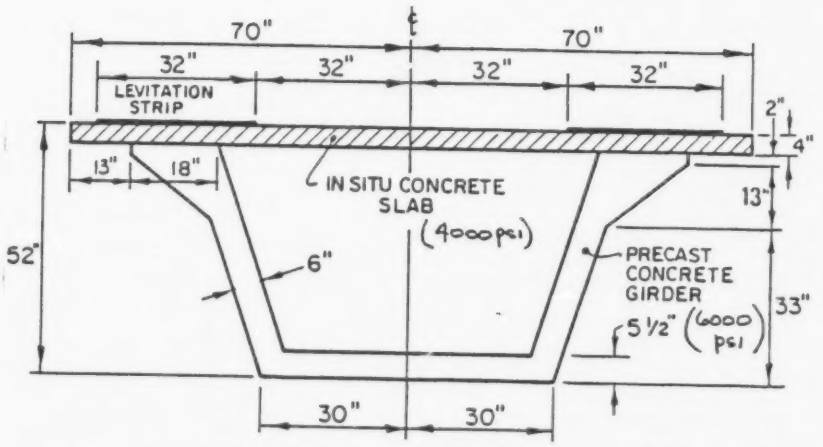
TEN/YR-PROD-PERIOD

11,50-BEAMS/PERIOD

6-MOLDS-REQ,D
6
0

6PCS/DAY-PROD

1175PCS/YR-PROD



TYPICAL CROSS SECTION

SPAN = 25 m

47,000 BEAMS

f_y = 60 KSI

f_{pu} = 250 KSI

U-SECT(fig1)

I TE ENRG.
1 *LENGTH(m)
25.00
2 DEV-AREA(sq m
118.00
3 VOLUME(cu m)
22.35
4 WEIGHT(m ton)
53.70

**U-SECT(fig1)

PRICE-Fob-PLANT-86
0

HOURLY-RATE:$15.25
0

($)/UNIT

LISTE ENRG.
1 *LABOUR
3,536.00
2 *MATERIALS
4,194.00
3 *ENG
100.00
4 O.H&PROF-42%
3,289.00
5 *8% FED TAX
890.00

TOTAL:

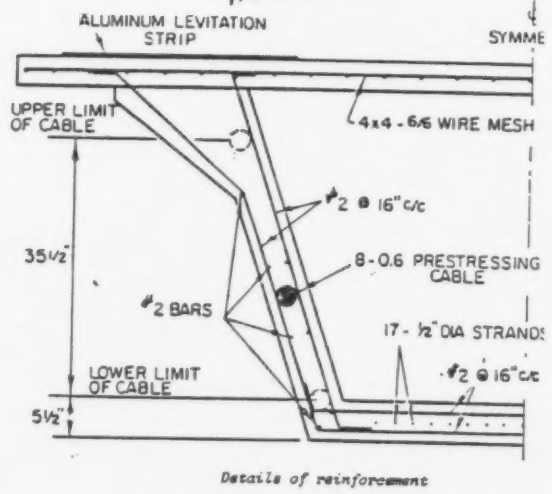
12,009.00

$12009/UNIT

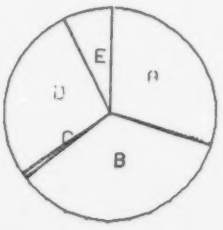
$480.36/LIN-METER

$537.31/CU-METER

PROV-TAX-EXTRA
-12009



Details of reinforcement



A *LABOUR
..... 29.44%
B *MATERIALS
..... 34.92%
C *ENG
..... 0.83%
D *O.H&PROF
..... 27.39%
E *8% FED TA
..... 7.42%

U-SECT(fig1)

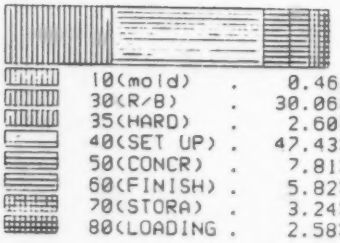
LABOUR($/UNIT)

LISTE ENRG.

1	*10(mold)sq m	232.00
2	*15sp form-hr	0.00
3	*30(R/B)Kg	1,610.00
4	*35(HARD)Kg	82.00
5	*40(setup)cu m	22.35
6	*50(conc)cu m	22.35
7	*60(finish)hr	13.50
8	*70(storag)hr	7.50
9	*80loading-hr	6.00

* DETAILS *

*10(mold)sq m		
@	232.00	
*	0.07	
=	16.24	
		16.24
*30(R/B)Kg		
@	1,610.00	
*	0.66	
=	1,062.60	
		1,062.60
*35(HARD)Kg		
@	82.00	
*	1.12	
=	91.84	
		91.84
*40(setup)cu m		
@	22.35	
*	75.03	
=	1,676.92	
		1,676.92
*50(conc)cu m		
@	22.35	
*	12.35	
=	276.02	
		276.02
*60(finish)hr		
@	13.50	
*	15.25	
=	205.87	
		205.87
*70(storag)hr		
@	7.50	
*	15.25	
=	114.37	
		114.37
*80loading-hr		
@	6.00	
*	15.25	
=	91.50	
		91.50
TOTAL		
		3,535.37



10(mold)	.	0.46%
30(R/B)	.	30.06%
35(HARD)	.	2.60%
40(SET UP)	.	47.43%
50(CONCR)	.	7.81%
60(FINISH)	.	5.82%
70(STORA)	.	3.24%
80(LOADING	.	2.58%

U-SECT(fig1)

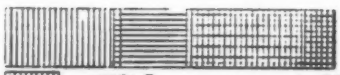
MATERIAL($/UNIT)

LISTE ENRG.

1	*MOLD(sq m)	232.00
2	*R/B (Kg)	1,610.00
3	*15mm STRAN(m	510.00
4	*L HOOKS(Qty)	4.00
5	galv HARD(Kg)	46.00
6	*50mm DUCT(m)	52.00
7	CONCRETE(cu m)	22.35
8	U/MAT(cu m)	22.35

* DETAILS *

*MOLD(sq m)		
@	232.00	
*	0.28	
=	64.96	
		64.96
*R/B (Kg)		
@	1,610.00	
*	0.51	
=	821.10	
		821.10
*15mm STRAN(m		
@	510.00	
*	0.79	
=	402.90	
		402.90
*L HOOKS(Qty)		
@	4.00	
*	10.00	
=	40.00	
		40.00
galv HARD(Kg)		
@	46.00	
*	1.41	
=	64.86	
		64.86
*50mm DUCT(m)		
@	52.00	
*	16.35	
=	850.20	
		850.20
CONCRETE(cu m)		
@	22.35	
*	67.11	
=	1,499.90	
		1,499.90
U/MAT(cu m)		
@	22.35	
*	20.13	
=	449.90	
		449.90
TOTAL		
		4,193.83



*MOLD	.	1.55%
*R/B	.	19.58%
*15mm STRA	.	9.61%
*L HOOKS	.	0.95%
*GALV-HARD	.	1.55%
*50mm DUCT	.	20.27%
*CONCRETE	.	35.76%
*U/MAT	.	10.73%